SIMPLIFIED DESIGN OF REINFORCED CONCRETE SLABS AND BEAMS

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A Thesis

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Under the Supervision of

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DECLARATION

The work performed in this thesis for the achievement of the degree of Bachelor of Science in Civil Engineering is “A study on Simplified Design of Reinforced Concrete Slabs and Beams”. The whole work is carried out by the authors under the strict and friendly supervision of Dr. Enamur Rahim Latifee, Associate Professor, Department of Civil Engineering, Ahsanullah University of Science and Technology, Dhaka, Bangladesh.

Neither this thesis nor any part of it is submitted or is being simultaneously submitted for any degree at any other institutions.

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TO OUR

BELOVED PARENTS AND TEACHERS

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ABSTRACT

Traditional method of building design includes several steps, which is often very lengthy and time consuming. It is sometimes very much complex to understand for the engineering students as well as young designers. So the concept of simplified building design is introduced. The main difference between simplified building design and traditional method of building design is the assumption of certain parameters. Design procedure includes several equations which contains certain parameters. In simplified design, some of the parameters are made constant, to shorten the design procedure and to make the design much more time efficient. The parameters like strength of reinforcement and properties of cement are made constant depending on availability of the material. This paper on "**Simplified Design of Reinforced Concrete Slabs and Beams**" was prepared by applying the provisions contained in ACI Standard 318, *Building Code Requirements for Structural Concrete*. The ACI 318 standard applies to all types of building uses; structures of all heights ranging from the very tall high-rise down to single-story buildings; facilities with large areas as well as those of nominal size. This paper has been written as a timesaving aid for use by those who consistently seek ways to simplify design procedures. The first part of this paper contains the design steps of one way slab along with an example. The design procedure includes determination of the thickness of the slab and its reinforcement detailing. There are several methods for determining the slab thickness. It can be determined by determining the value of α (the relative stiffness of the beam and slab spanning in either direction). Thickness can also be determined by using the formula $\frac{\text{perimeter}}{145}$. The design steps for simplified beam design are also included in the paper. This paper also includes minimum and maximum number of bars in a single layer for beams of various widths. In case of beam stirrups, the selection and spacing of stirrups can be simplified if the spacing is expressed as a function of the

effective depth. A comparison table is developed which includes the design live loads for various occupancies suggested by different codes. In simplified design, design load & live load are considered in accordance of the code. If wind load, resistance to earthquake, induced forces, earth or liquid pressure, impact effects or structural effects of differential settlement need to be included in the design, such effects should be considered separately.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF SYMBOLS AND ABBREVIATIONS

ACI = American Concrete Institute.

ASCE = American Society of Civil Engineers.

BNBC = Bangladesh National Building Code.

IBC = International Building Code.

UBC = Uniform Building Code.

EN = Euro/British Code.

DDM =Direct Design Method.

EFM = Equivalent Frame Method.

DL = Dead load.

LL = Live load.

l = Length of span.

l_n = Length of clear span.

l_A = Length of clear span in short direction.

l_B = Length of clear span in long direction.

A_s = Area of steel.

f'_c = Compressive strength of concrete.

f_y = Grade/Yield strength of steel.

h = Thickness of slab.

β = Ratio of clear span in long direction to clear span in short direction.

α = Stiffness ratio of beam to slab.

E = Modulus of elasticity.

I = Moment of inertia.

R^2 = Least Squares.

P = Perimeter.

ρ = Steel ratio.

Chapter 01
INTRODUCTION

This paper was prepared for the purpose of suggesting possible ways of reducing design time in applying the provisions contained in the ACI 318-11 *Building Code Requirements for Structural Concrete.* The ACI 318 standard applies to all types of building uses; structures of all heights ranging from the very tall high-rise down to single-story buildings; facilities with large areas as well as those of nominal size; buildings having complex shapes and those primarily designed as uncomplicated boxes; and buildings requiring structurally intricate or innovative framing systems in contrast to those of more conventional or traditional systems of construction. The general provisions developed to encompass all these extremes of building design and construction tend to make the application of ACI 318 complex and time consuming. However, this need not necessarily be the case, as is demonstrated in the paper. This paper has been written as a timesaving aid for use by those who consistently seek ways to simplify design procedures.

A complex code is unavoidable since it is necessary to address simple and complex structures in the same document. The purpose of this paper is to give practicing engineers some way of reducing the design time required for smaller projects, while still complying with the letter and intent of the ACI Standard 318, *Building Code Requirements for Structural Concrete*. The simplification of design with its attendant savings in design time result from avoiding building member proportioning details and material property selections which make it necessary to consider certain complicated provisions of the ACI Standard. These situations can often be avoided by making minor changes in the design approach. The simplified design procedures presented in this manual are an attempt to satisfy the various design considerations that need to be addressed in the structural design and detailing of primary framing members of a reinforced concrete building by the simplest and quickest procedures possible. The simplified design material is intended for use by those well versed in the design principles of reinforced concrete and completely familiar with the design provisions of ACI 318. It aims to arrange the information in the code in an organized step-by-step procedure for the building and member design. The formulae and language avoid complicated legal terminology without changing the intent or the objective of the code. As noted above, this paper is written solely as a time saving design aid; that is, to simplify design procedures using the provisions of ACI 318 for reinforced concrete buildings of moderate size and height.

Structural design is the set of decisions, inventions, and plans that results in a fully described structure, ready for construction. To get to that fully described structure in a practical manner is the designer's pragmatic task. To achieve that task, designers typically use information from previous design, from observations of previously built structures, from result of research, and from the general body of publications that record to collective experience of concrete construction. Invention, innovation, and experimentation help to advance knowledge, but experience provides the confidence to trust our design practices.

The materials in this paper are not intended for well-trained, experienced structural engineers but rather for people who are interested in the topic but lack both training and experience in structural design. With this readership in mind, the computational work here is reduced to a minimum, using mostly simple mathematical procedures. A minimum background for the reader is assumed in fundamentals of structural mechanics.

The design work presented here conforms in general with the 2011 edition of *Building Code Requirements for Structural Concrete*, ACI 318-11, which is published by the American Concrete Institute and is commonly referred to as the ACI Code. For general reference, information is used from *Minimum Design Loads for Buildings and Other Structures*, SEI/ASCE 7-02, published by the American Society of Civil Engineers, from the 1997 edition of the *Uniform Building Code, Volume 2: Structural Engineers Provisions*, published by the International Conference of Building Officials, and from the 2007 edition of the *Bangladesh National Building Code*, is commonly referred to as the BNBC. For any actual design work, however, the reader is cautioned to use the codes currently in force in the location of the construction.

There are a lot of commercial softwares available to solve structural design problems such as ETABS, STAAD Pro, etc. However, to use these softwares, a person must have some basic civil engineering knowledge and it also requires detail information input for modeling. The engineers, architects and ordinary people including home owner, contractors and others do not have any handy software available to get very quick design and estimation of materials. Moreover, no unpaid tool is available for them to have an idea of the size and cost of building elements, such as slab, beam thickness, total steel by weight etc. and even paid softwares need detail data input. To overcome these shortcomings of the modern day softwares and to empower the engineers, architects and others to have structural design and estimated materials ready with very simple inputs; a web based, free of cost application is developed according to ACI-318-11 with visual output. The user including the engineer, the architect, and common non-technical person can give very simple inputs (e.g. slab width, length etc.) in a webpage and instantly get the visual results there. It can be used for initial structural design, verifying existing design and detail estimation of materials. There are also a lot of methods for design a reinforced concrete building, but here we used simplified design method of reinforced concrete buildings.

The aim is to create a simplification in the design process with minimum user input. The design aids in the form of graphs, are being generated for regular cases-residential/commercial. The scope is limited to moderate size and height of buildings. A website has been created to benefit the user- free to use.

Simplified Web based online Reinforced Concrete Structure Design Tools is a JavaScript language based online programming webpage- free to be used online or offline, in case one wants to save the web page. No login required there. Some designs tools have been done in Microsoft office excel worksheet and uploaded to internet via Google drive. So one can easily use those tools in internet without installing Microsoft office in their computer.

Since we started our thesis by simplification of one way slab, but till now we have completed one way slab, two way slab and beam design (flexural and shear) for both simplified design method and also for simplified web based online reinforced concrete building design tools. We hope to cover the full building solution in near future.

Chapter 02
LITERATURE REVIEW

2.1 One way slab

A rectangular reinforced concrete slab which spans a distance very much greater in one direction than the other; under these conditions, most of the load is carried on the shorter span. This type of slab is called one way slab.

2.1.1 Design steps of one way slab

Step 01: Determination of minimum slab thickness.

Determine minimum slab thickness h (in), which depends on length of span l (in) and support condition, according to Table 2.1.

Table 2.1: Minimum thickness h of nonprestressed one way slabs. (ACI Code 318-11 section 9.5.2, Table 9.5 (a))

Support Condition	**Minimum thickness, h** (in)
Simply supported	l/20
One end continuous	l/24
Both ends continuous	l/28
Cantilever	l/10

Note 2.1: If the slab rests freely on it's supports, the span length may be taken equal to the clear span plus the depth of the slab but need not exceed the distance between centers of supports, according to ACI Code 8.7.1. Slab thickness is rounded to next higher 1/4 in. for slab up to 6 in. thickness. The concrete portion below the reinforcement should the requirements of ACI Code 7.7.1, calling for 3/4 in. below the bottom of the steel.

Step 02: Calculation of factored load.

W_{DL}= 1.2*DL and W_{LL}= 1.6*LL;

W_u= W_{DL}+W_{LL}

Where DL= Total dead load (i.e.: Slab self weight, Floor finish, Partition wall, Plaster etc.)

LL= Live load.

Step 03: Determination of factored moments.

Factored moment coefficient found in Table 2.2.

Table 2.2: Moment and shear values using ACI coefficients. (ACI Code 318-11, section 8.3.3)

Positive moment	
End spans	
If discontinuous end is unrestrained	$\frac{1}{11} w_u l_n^2$
If discontinuous end is integral with the support	$\frac{1}{14} w_u l_n^2$
Interior spans	$\frac{1}{16} w_u l_n^2$
Negative moment at exterior face of first interior support	
Two spans	$\frac{1}{9} w_u l_n^2$
More than two spans	$\frac{1}{10} w_u l_n^2$
Negative moment at other faces of interior supports	$\frac{1}{11} w_u l_n^2$
Negative moment at face of all supports for (1) slabs with spans not exceeding 10 ft and (2) beams and girders where ratio of sum of column stiffness to beam stiffness exceeds 8 at each end of the span	$\frac{1}{12} w_u l_n^2$
Negative moment at interior faces of exterior supports for members built integrally with their supports	
Where the support is a spandrel beam or girder	$\frac{1}{24} w_u l_n^2$
Where the support is a column	$\frac{1}{16} w_u l_n^2$
Shear in end members at first interior support	$1.15 \frac{w_u l_n}{2}$
Shear at all other supports	$\frac{w_u l_n}{2}$

† w_u = total factored load per unit length of beam or per unit area of slab.
l_n = clear span for positive moment and shear and the average of the two adjacent clear spans for negative moment.

Step 04: Determination of steel area, A_s

$A_s = \frac{M_u}{\emptyset f_y (d - \frac{a}{2})}$ Where, Ø= 0.9 for flexure design.

Checking the assumed depth, $a = \frac{A_s f_y}{0.85 f' cb}$

According to ACI Code 318-11, section 13.3.1 the minimum reinforcement in each direction shall be as mentioned below:

For 40 grade rebar: $A_{s\,min} = 0.0020 \times b \times h$

60 grade rebar: $A_{s\,min} = 0.0018 \times b \times h$

$>$ 60 grade rebar: $A_{s\,min} = \frac{0.0018 \times 60{,}000}{f_y} \times b \times h$

Step 05: Determining the spacing of the steel bars

$$\text{Spacing} = \frac{\text{area of the bar used} * \text{width of the strip}}{\text{required steel area}}$$

Note 2.2: The lateral spacing of the bars, except those used to control shrinkage and temperature cracks should not exceed 3 times the thickness h or 18 inch, whichever is less, according to ACI Code 318-11, section 7.6.5. Generally, bar size should be selected so that the actual spacing is not less than about 1.5 times the slab thickness, to avoid excessive cost for bar fabrication & handling. Also to reduce cost, straight bars are usually used for slab reinforcement.

Note 2.3: Temperature and shrinkage reinforcement: Reinforcement for shrinkage and temperature stresses normal to the principal reinforcement should be provided on a structural slab in which the principal reinforcement extends in one direction only. ACI Code 318-11, section 7.12.2 specifies the minimum ratios of reinforcement area to gross concrete area (i.e. based on the total depth of the slab), but in no case may such reinforcing bars be placed farther apart than 5 times the slab thickness or more than 8 inch. In no case is the reinforcement ratio to be less than 0.0014.

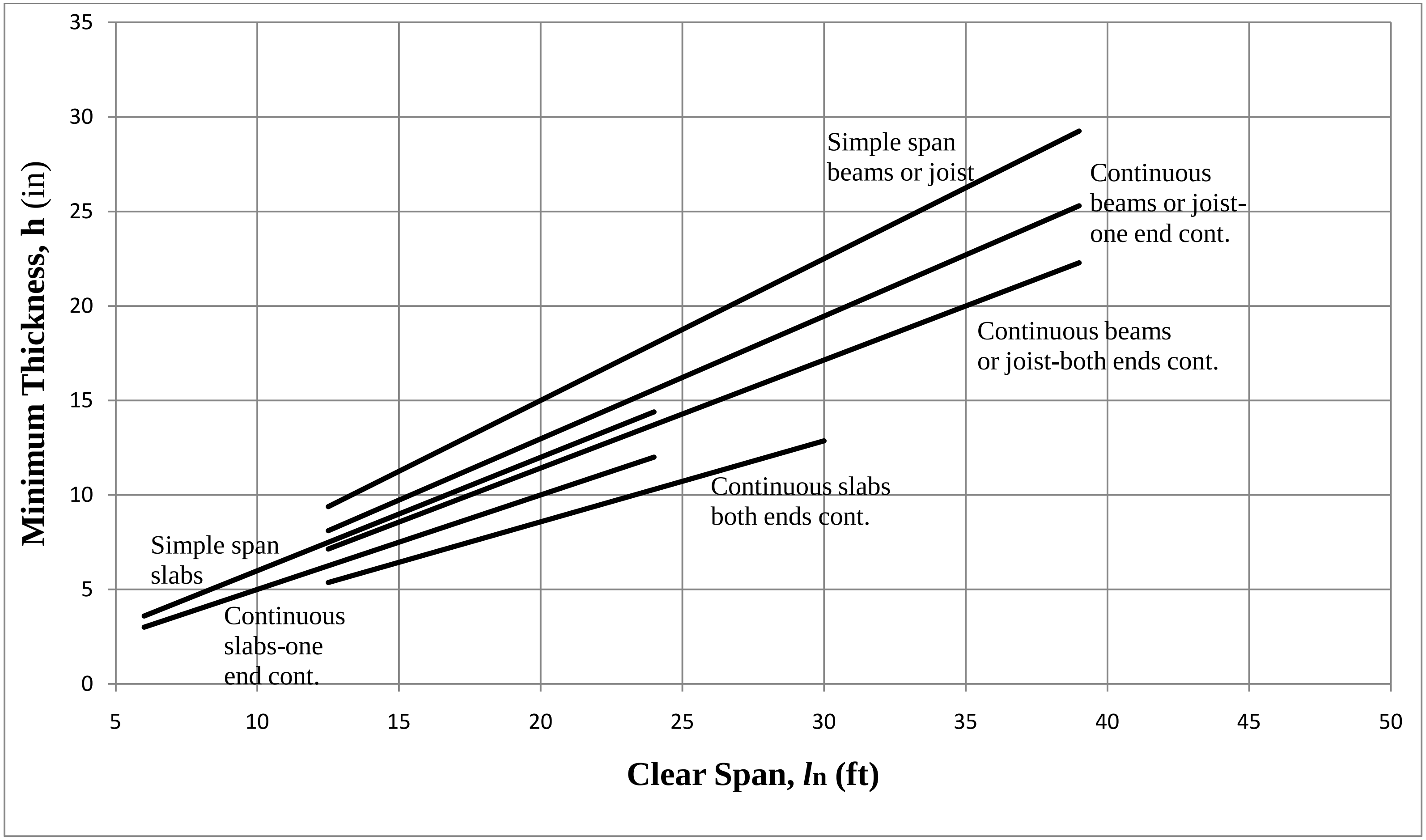


Figure 2.1: Minimum Thicknesses for Beams and One-Way Slabs.[2.1]

2.2 Two way slab

Two way slabs are the slabs that are supported on four sides and the ratio of longer span (B) to shorter span (A) is less than 2. In two way slabs, load will be carried in both the directions. So, main reinforcement is provided in both directions for two way slabs.

For two way slab design, ACI supported two methods. Those are:

(1) Direct design method (DDM) (ACI Code 318-11, section 13.6)

Note 2.3: The direct design method consists of a set of rules for distributing moments to slab and beam sections to satisfy safety requirements and most serviceability requirements simultaneously.

(2) Equivalent frame method (EFM) (ACI Code 318-11, section 13.7)

Note 2.4: The equivalent frame method involves the representation of the three-dimensional slab system by a series of two-dimensional frames that are then analyzed for loads acting in the plane of the frames. The negative and positive moments so determined at the critical design sections of the frame are distributed to the slab sections in accordance with 13.6.4 (column strips), 13.6.5 (beams), and 13.6.6 (middle strips). The equivalent frame method is based on studies reported in Reference 13.18, 13.19, and 13.20 (ACI Code 318-11). Many of the details of the equivalent frame method given in the Commentary in the 1989 Code were removed in the 1995 Code.

There was another design method named “ACI moment coefficient method”. The moment coefficient method included for the first time in 1963 ACI Code is applicable to two way slabs supported on four sides of each slab panels by walls, steel beams. However it was discontinued in 1977 and later versions of ACI Code.

But BNBC published in 2013 (draft) included this method and supporting this method named “Alternative design method of two way edge supported slabs”.

2.2.1 Determination of thickness of slab

The parameter used to define the relative stiffness of the beam and slab spanning in either direction is α, calculated from $\alpha = \frac{\mathbf{E}_{cb}\,\mathbf{I}_b}{\mathbf{E}_{cs}\,\mathbf{I}_s}$. In which $\mathbf{E}_{cb}$ and $\mathbf{E}_{cs}$ are the modulus of elasticity of the beam and slab concrete (usually the same) and $\mathbf{I}_b$ and $\mathbf{I}_s$ are the moments of inertia of the effective beam and the slab. Then α_m is defined as the average value of α for all beams on the edges of a given panel. According to ACI code 9.5.3.3, for α_m equal to or less than 0.2, the minimum thickness of Table 2.3 shall apply.

Table 2.3: Minimum thickness of slabs without interior beams (ACI 318-11 Table 9.5(c))

	Without Drop Panels			With Drop Panels		
Yield Stress f_y, psi	**Exterior Panels**		**Interior Panels**	**Exterior Panels**		**Interior Panels**
	Without Edge Beams	**With Edge Beams**[a]		**Without Edge Beams**	**With Edge Beams**[a]	
40,000	$l_n/33$	$l_n/36$	$l_n/36$	$l_n/36$	$l_n/40$	$l_n/40$
60,000	$l_n/30$	$l_n/33$	$l_n/33$	$l_n/33$	$l_n/36$	$l_n/36$
75,000	$l_n/28$	$l_n/31$	$l_n/31$	$l_n/31$	$l_n/34$	$l_n/34$

[a] Slabs with beams along exterior edges. The value of α for the edge beam shall not be less than 0.8.

For α_m greater than 0.2 but not greater than 2.0, the slab thickness must not be less than

$$h=\frac{l_n\left(0.8+\left(\frac{fy}{200{,}000}\right)\right)}{36+5\beta\left(\alpha_m-0.2\right)}$$ and not less than 5.0 inch….. (2.1) (ACI Eq. 9-12)

For α_m greater than 2.0, the thickness must not be less than

$$h=\frac{l_n\left(0.8+\left(\frac{fy}{200{,}000}\right)\right)}{36+9\beta}$$ and not less than 3.5 inch….. (2.2) (ACI Eq. 9-13)

Where l_n = clear span in long direction, in.

α_m = average value of α for all beams on edges of a panel. [$\alpha=\frac{\mathbf{E_{cb}}\,\mathbf{I_b}}{\mathbf{E_{cs}}\,\mathbf{I_s}}$]

β = ratio of clear span in long direction to clear span in short direction.

At discontinuous edges, an edge beam must be provided with a stiffness ratio α not less than 0.8; otherwise the minimum thickness provided by Eq. (2.1) or (2.2) must be increased by at least 10 percent in the panel with the discontinuous edge.

In all cases, slab thickness less than stated minimum may be used if it can be shown by computation that deflections will not exceed the limit values of Table 2.4.

Table 2.4: Maximum allowable computed deflections. (ACI 318-11 Table 9.5(b))

<table>
<tr><th>Type of member</th><th>Deflection to be considered</th><th>Deflection Limitation</th></tr>
<tr><td>Flat roofs not supporting or attached to nonstructural elements likely to be damaged by large deflections</td><td>Immediate deflection due to the live load L</td><td>$\frac{l}{180}$</td></tr>
<tr><td>Floors not supporting or attached to nonstructural elements likely to be damaged by large deflections</td><td>Immediate deflection due to the live load L</td><td>$\frac{l}{360}$</td></tr>
<tr><td>Roof or floor construction supporting or attached to nonstructural elements likely to be damaged by large deflections</td><td rowspan="2">That part of the total deflection occurring after attachment of the nonstructural elements (sum of the long-time deflection due to all sustained loads and the immediate deflection due to any additional live load)</td><td>$\frac{l}{480}$</td></tr>
<tr><td>Roof or floor construction supporting or attached to nonstructural elements not likely to be damaged by large deflections</td><td>$\frac{l}{240}$</td></tr>
</table>

Equations (2.1) and (2.2) can be rested in the general form

$$h=\frac{l_n\,(0.8+\left(\frac{f_y}{200{,}000}\right))}{F} \quad \text{….. (2.3)}$$

Where F is the value of the denominator in each case. Figure 2.2 shows the value of F as a function of α_m, for comparative purposes, for three panel aspect ratios β:

1. Square panel, with β= 1.0

2. Rectangular panel, with β= 1.5

3. Rectangular panel, with β= 2.0, the upper limit of applicability of Equations (2.1) and (2.2)

Note that, for α_m less than 0.2, column-line beams have little effect, and minimum thickness is given by Table 2.3. For stiff, relatively deep edge beams, with α_m of 2 or greater, Eq. (2.2) governs. Equation (2.1) provides a transition for slabs with shallow column-line beams having α_m in the range from 0.2 to 2.0.

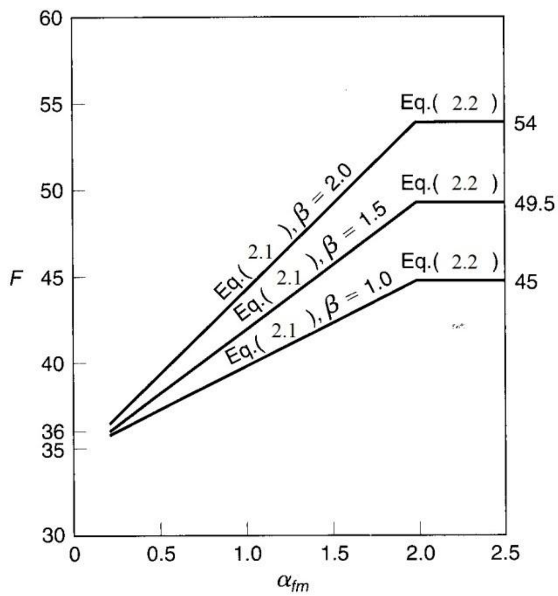


Figure 2.2: Parameter F governing minimum thickness of two-way slabs; minimum

thickness h= $\frac{l_n (0.8+\left(\frac{fy}{200,000}\right))}{F}$

2.2.2 Calculation of alpha

We know $\alpha = \frac{\mathbf{E_{cb}\ I_b}}{\mathbf{E_{cs}\ I_s}}$. Here $E_{cb} = E_{cs}$. Because of beam and slab concrete is same. So we can write $\alpha = \frac{\mathbf{I_b}}{\mathbf{I_s}}$.

<u>Calculation of I_b and I_s:</u>

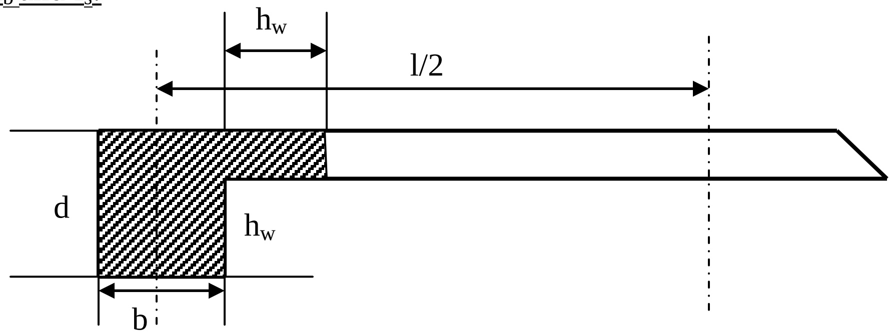


Figure 2.3 (a): Section for I_b – Edge beam.

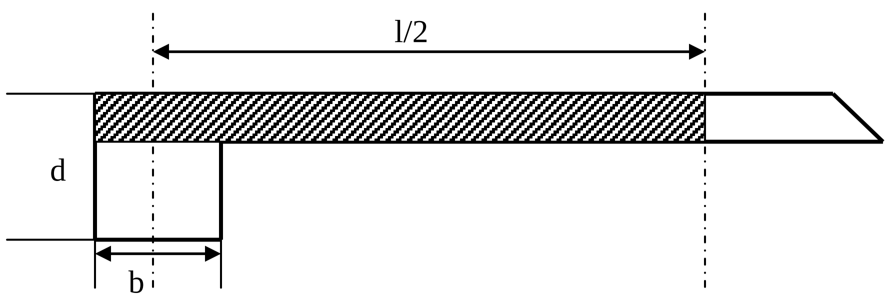


Figure 2.3 (b): Section for I_s – Edge beam.

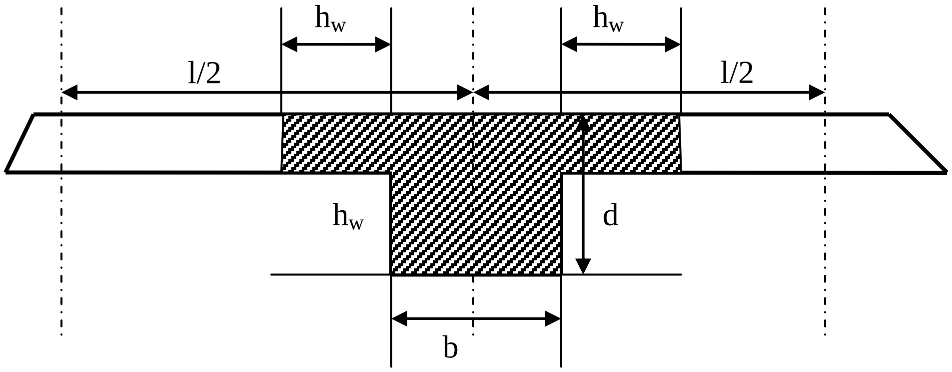


Figure 2.3 (c): Section for I_b – Interior beam.

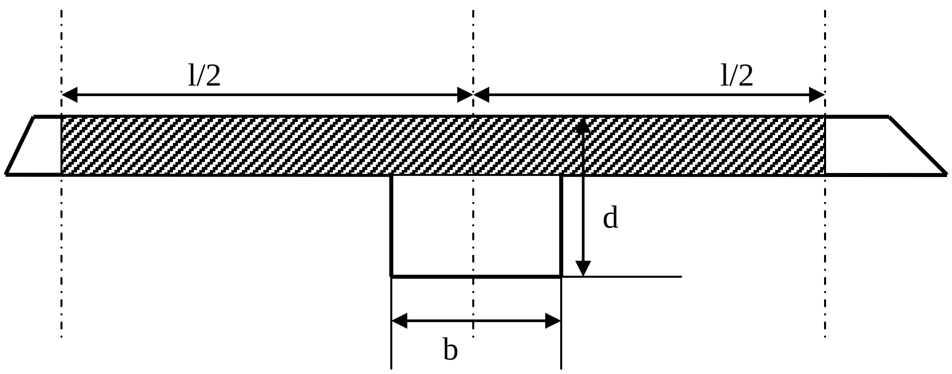


Figure 2.3 (d): Section for I_s – Interior beam.

In which $\mathbf{E}_{cb}$ and $\mathbf{E}_{cs}$ are the modulus of elasticity of the beam and slab concrete (usually the same) and $\mathbf{I}_b$ and $\mathbf{I}_s$ are the moments of inertia of the effective beam and the slab. Then α_m is defined as the average value of α for all beams on the edges of a given panel.

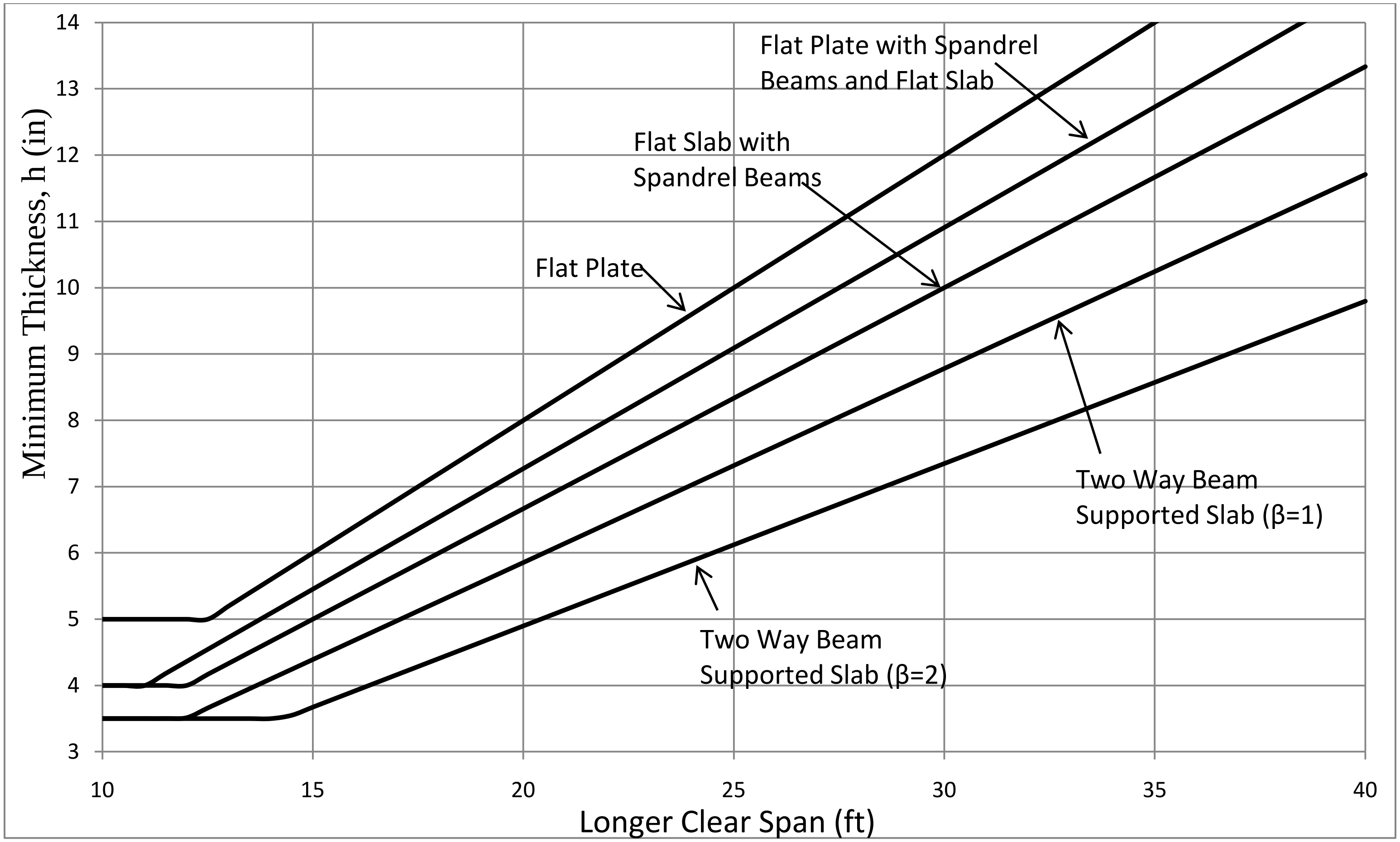


Figure 2.4: Minimum Slab Thickness for Two-Way Slab Systems.[2.2]

2.3 Beam

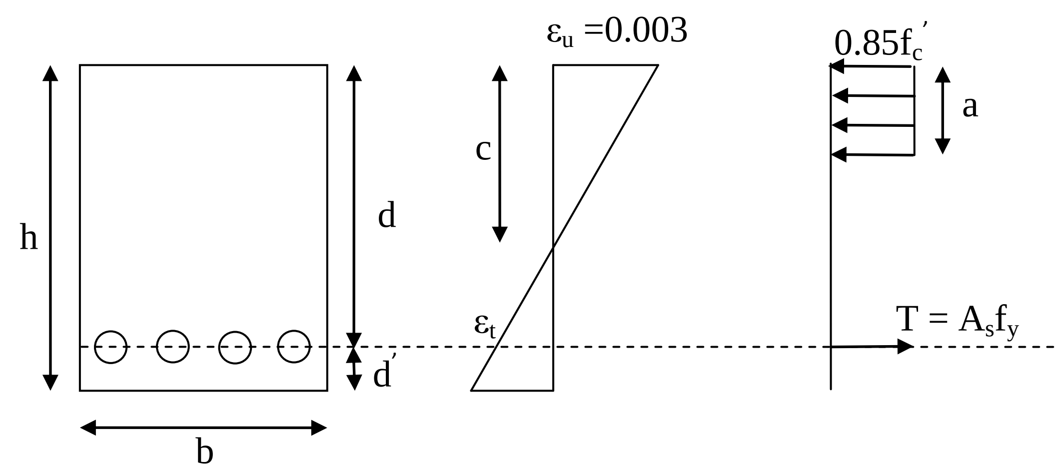
2.3.1 Simplified design of beam

Beam design refers to optimization of beam cross section (i.e. width & depth) and reinforcement satisfying the design criteria. For beam, design criteria are serviceability and strength. Serviceability condition limits the deflection and strength resist the possible loads. In this study ACI 318 code is used for formulation of background theory.

Load Calculation:

To design a beam maximum bending moments at mid span and supports are required. Moments can be calculated from exact analysis (i.e. moment distribution, slope deflection method etc) using finite element software package. But approximate bending moments can be calculated from ACI 318 section 8.3.3. Moment coefficients are given in the Table 2.2.

Ductile design concept:



a) Cross Section

b) Strain Compatibility

c) Equivalent Rectangular Stress Distribution

Ultimate flexural strength of a beam can be reached when compression concrete collapse. At collapse, strain of tension steel may equal of yield strain or more or less than yield strain depending on the amount of reinforcement (steel ratio) and material properties. A tension failure initiated by yielding of steel is preferred for under-reinforced beam design as it provides gradual warning before collapse. To ensure tension failure ACI 318 section 10.3.5 limits the reinforcement ratio to a certain value based on net tensile strain. The minimum net tensile strain ε_t at the nominal member strength of 0.004 for members subjected to axial loads less than $0.10f_c^{'}A_g$, where A_g is the gross section of the cross section. It is generally most economical to design beams such that the strain in the extreme layer of tension reinforcement exceeds 0.005.

From strain compatibility-

$$\frac{\varepsilon_u}{c} = \frac{\varepsilon_t}{d-c}$$

$$\Rightarrow \quad c = \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} d$$

From equilibrium-

$$C = T$$

$$\Rightarrow \quad 0.85f_c^{'}ab = A_s f_y$$

$$\Rightarrow \quad 0.85f_c^{'}(\beta_1 c)b = \rho b d f_y, \quad \text{Where } \beta_1 = 0.85 - 0.05\,\frac{f_c^{'} - 4000}{1000}$$

$$\Rightarrow \quad \rho = 0.85\beta_1 \frac{f_c^{'}}{f_y} \frac{c}{d}$$

$$\Rightarrow \quad \rho = 0.85\beta_1 \frac{f_c^{'}}{f_y} \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t}$$

$$\Rightarrow \quad \rho_{max} = 0.85\beta_1 \frac{f_c^{'}}{f_y} \frac{\varepsilon_u}{\varepsilon_u + 0.004}$$

For $f_c^{'}$ = 3000 psi and f_y = 60000 psi, ρ_{max} = 0.0155

For economical and ductile design one can assume ρ corresponding to 0.005 tensile strain at outer layer of tension steel or less. Steel ratios for 0.005 tensile strain for different material properties are given in Table 2.5.

Table 2.5: Steel ratio (ρ) for 0.005 tensile strain at outer tension steel layer.

Concrete Strength (ksi)	**Steel Grade (ksi)**	
	60	**72.5 (500 W)**
3	0.013547	0.01121
3.5	0.015805	0.01308
4	0.018063	0.01495
5	0.02125	0.01759
6	0.023906	0.01978
7	0.026031	0.02154
8	0.027625	0.02286

Member sizing:

Cross sectional dimension of a beam to resist maximum bending moment can be determined from simple mechanics considering equivalent rectangular stress block according to ACI 318 section 10.2.7.

From equilibrium condition-

$$C = T$$

⇨ $0.85f_c' ab = A_s f_y$

⇨ $a = \dfrac{A_s f_y}{0.85 f_c' b}$

⇨ $a = \dfrac{\rho d f_y}{0.85 f_c'}$

Now Design moment, $M_u = \varphi A_s f_y \left(d - \dfrac{a}{2}\right)$

So, $M_u = \varphi \rho b d f_y \left(d - \dfrac{0.5 \rho d f_y}{0.85 f_c'}\right)$

⇨ $M_u = \varphi \rho b d^2 f_y \left(1 - \dfrac{0.5 \rho f_y}{0.85 f_c'}\right)$

⇨ $\dfrac{M_u}{\varphi b d^2} = \rho f_y \left(1 - \dfrac{0.5 \rho f_y}{0.85 f_c'}\right)$

⇨ $\dfrac{M_u}{\varphi b d^2} = R_n$

So, $(bd^2)_{req} = \dfrac{M_u}{\varphi R_n}$

For, $f_c' = 3000$ psi, $f_y = 60000$ psi and $\rho = 0.0125$

So, $R_n = 0.0125 \text{ x } 60000 \left(1 - \dfrac{0.5 \text{ x } 0.0125 \text{ x } 60000}{0.85 \text{ x } 3000}\right) = 639.71$ psi

Now, $(bd^2)_{req} = \dfrac{M_u}{\varphi R_n} = \dfrac{Mu \text{ x } 12 \text{ x } 1000}{0.9 \text{ x } 639.71}$

$(bd^2)_{req} = 21\ M_u$, Where M_u in K-ft, b and d in inch.

This is a simplified equation for member sizing considering a certain steel percentage and material properties. Therefore this approach can be used to derive equations for different material properties and steel percentage. To generalize the equation can be written in form of-

$$\mathbf{(bd^2)_{req} = (K_1)\, M_u}, \quad \text{Where } M_u \text{ in K-ft, b and d in inch.}$$

Figure 2.5 (a) and Figure 2.5 (b) shows the design chart of member sizing for different steel percentage and material properties.

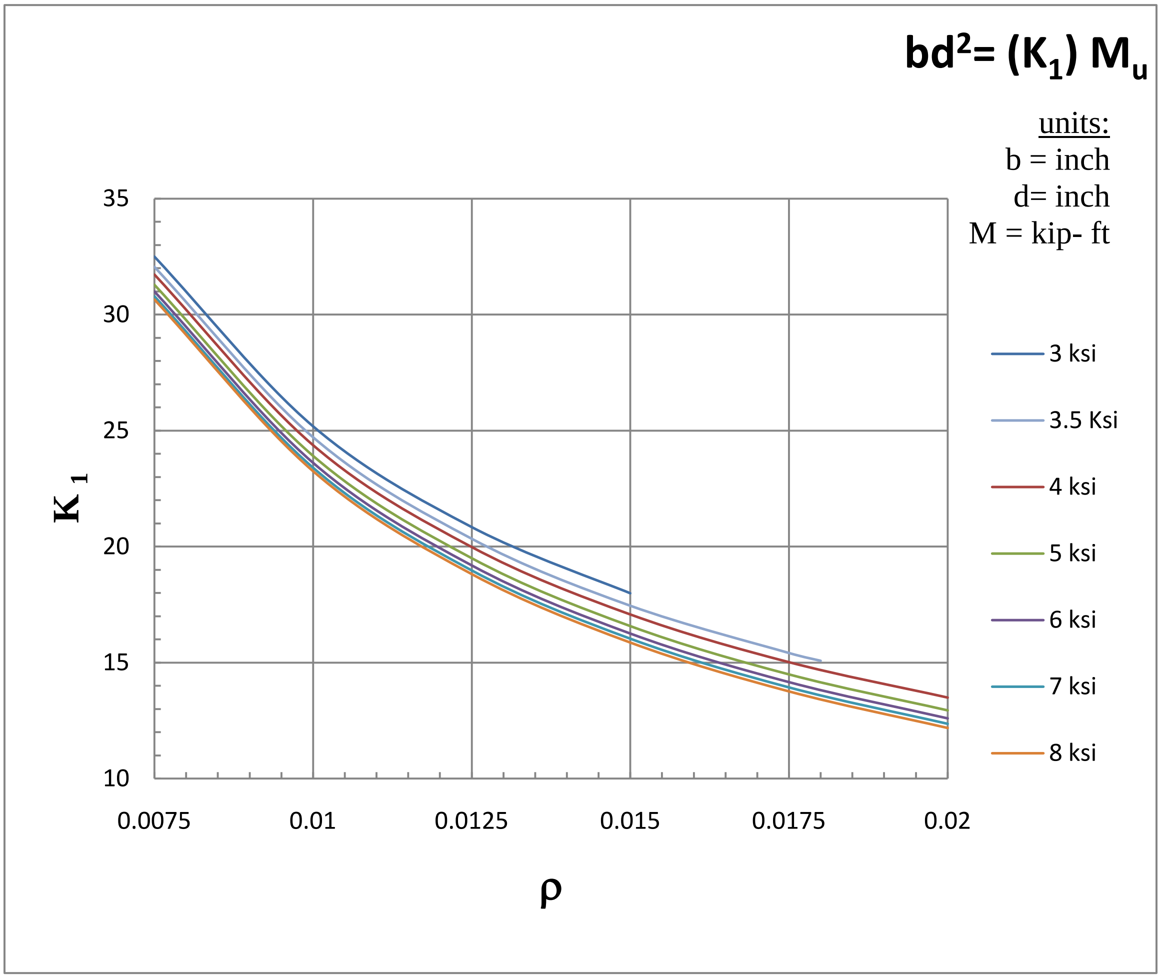


Figure 2.5 (a): Design chart for member sizing (f_y = 60 ksi).

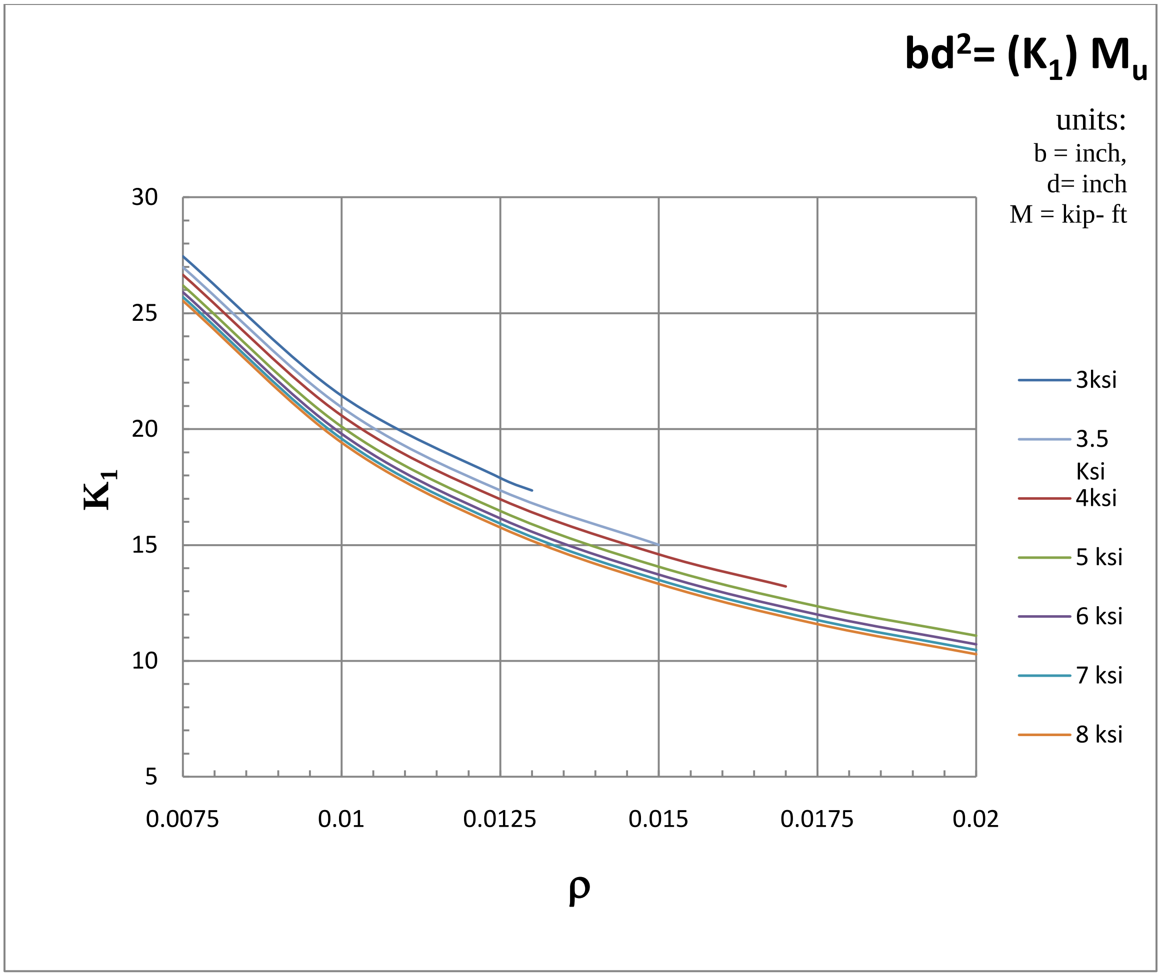
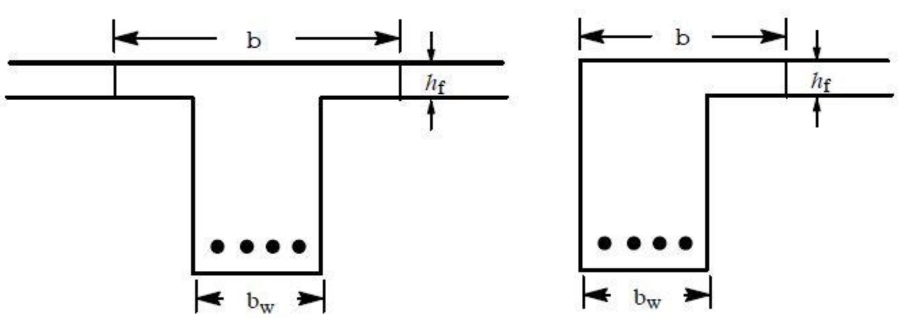


Figure 2.5 (b): Design chart for member sizing (f_y = 72.5 ksi).



T beam **L beam**

As slab and beams are casted monolithically it is permitted to include the contribution of the slab in beam. Effective width of the flange can be calculated as per ACI 318 section

8.10.2 which is given in the following table.

Table 2.6: Effective flange width.

T-Beam	**L-Beam**
$b \le \frac{\text{Span}}{4}$ $b - b_w \le 16h_f$ $b - b_w \le \frac{\text{C/C beam distance}}{2}$	$b - b_w \le \frac{\text{Span}}{12}$ $b - b_w \le 6h_f$ $b - b_w \le \frac{\text{C/C beam distance}}{2}$

Beam section should be designed adequately to limit the deflection that affects the serviceability of structure adversely. According to ACI 318 section 9.5.2.1 minimum thickness of beams are provided in Table 2.7.

Table 2.7: Minimum thickness of nonprestressed beams. (ACI Table 9.5 (a))

	Simply supported	**One end continuous**	**Both ends continuous**	**Cantilever**
Minimum thickness	$\frac{l}{16}$	$\frac{l}{18.5}$	$\frac{l}{21}$	$\frac{l}{8}$

Flexural Reinforcement:

To derive simplified equation of required reinforcement (A_s), linear relationship between R_n and ρ has been assumed though the relationship is not perfectly linear according to the equation, $R_n = \rho f_y \left(1-\frac{0.5\rho f_y}{0.85f_c'}\right)$. This assumption is valid up to about two-third of the maximum ρ.

Here, $\frac{M_u}{\varphi bd^2} = \rho f_y (1 - \frac{0.5\rho f_y}{0.85 f_c'})$

$$\Rightarrow \frac{M_u}{\varphi d} = \rho bd f_y (1 - \frac{0.5\rho f_y}{0.85 f_c'})$$

$$\Rightarrow \frac{M_u}{\varphi d} = A_s f_y (1 - \frac{0.5\rho f_y}{0.85 f_c'})$$

$$\Rightarrow A_s = \frac{M_u}{\varphi d\, f_y (1 - \frac{0.5\rho f_y}{0.85 f_c'})}$$

For, $f_c' = 3000$ psi, $f_y = 60000$ psi and $\rho = 0.0125$

$$A_s = \frac{M_u \text{ x}12 \text{ x } 1000}{0.9 \text{ x } d \text{ x } 60000 \, (1 - \frac{0.5 \text{ x } 0.0125 \text{ x } 60000}{0.85 \text{ x } 3000})}$$

$A_s = \frac{M_u}{3.85d}$, Where M_u in K-ft, d in inch and A_s in sq. inch.

This is a simplified equation for steel area considering a certain steel percentage and material properties. This approach can be used to derive equations for different material properties and steel percentage. To generalize the equation can be written in form of-

$$\mathbf{A_s = \frac{M_u}{d(K_2)}}$$, Where M_u in K-ft, d in inch and A_s in sq. inch.

Figure 2.6 (a) and Figure 2.6 (b) shows the design chart of steel area for different steel percentage and material properties.

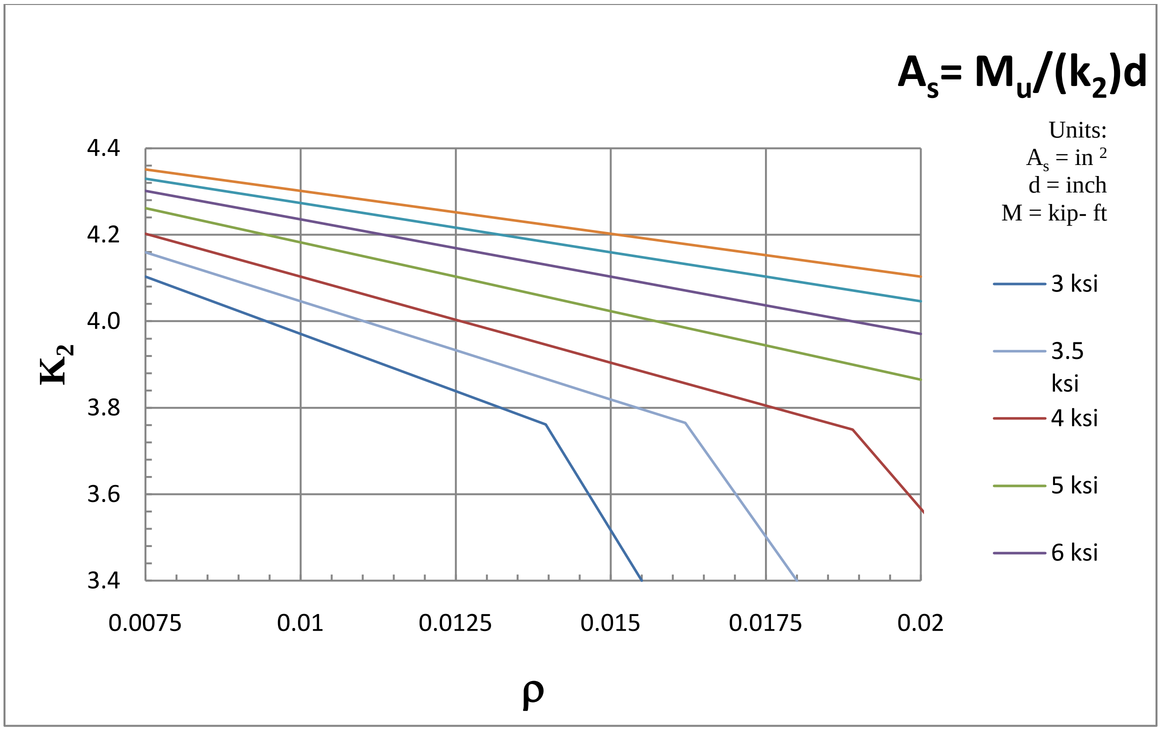


Figure 2.6 (a): Design chart for steel area (f_y = 60 ksi).

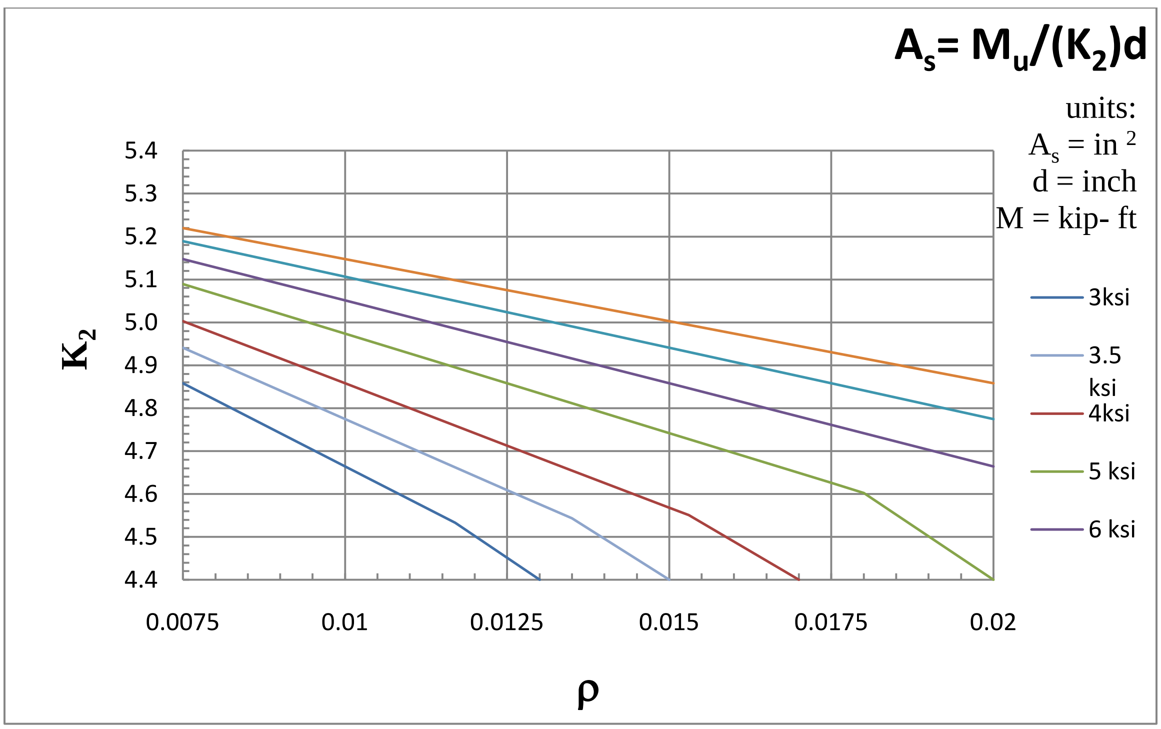


Figure 2.6 (b): Design chart for steel area (f_y = 72.5 ksi).

According to ACI 318 section 10.5 minimum tensile reinforcement should be provided to resist the cracking moment. For any section minimum reinforcement can be calculated by the equation-

$$(A_s)_{min} = \frac{3\sqrt{f_c'}}{f_y} b_w d \geq \frac{200}{f_y} b_w d, \quad \text{Where } f_c' \text{ and } f_y \text{ are in psi.}$$

Design steps:

- Calculate the ultimate moment (M_u) from Table 2.2 or exact calculation.
- Select preliminary ρ from Table 2.5 to ensure ductile behavior.
- Calculate effective b for T beam from Table 2.6.
- Pick constant K_1 from Figure 2.5 (a) or 2.5 (b) depending on the material properties.
- Calculate $(bd^2)_{req}$ using equation $(bd^2)_{req} = (K_1)\, M_u$.
- Calculate required d.
- Check for minimum d for serviceability (h) from Table 2.7.
- Provide governing d (larger of d_{req} , d_{min}).
- Take governing d.
- Pick constant K_2 from Figure 2.6 (a) or 2.6 (b) depending on the material properties.
- Calculate A_s using equation $A_s = \dfrac{M_u}{d(K_2)}$.
- Find $(A_s)_{min}$.
- Take governing A_s (larger of $(A_s)_{req}$, $(A_s)_{min}$).

Economical member sizing notes:

- Beam dimensions should be rounded to whole number.
- Beam width should be multiple of 2 or 3 in inch.
- Change amount of reinforcement instead of cross section for a continuous beam.
- Prefer wide flat beam rather narrow deep beam. Beam width should be equal or greater than column dimensions.

2.3.2 Minimum and maximum number of bars in a single layer for beams of various widths

Tables 2.8 and 2.9 give the minimum and maximum number of bars in a single layer for beams of various widths; selection of bars within these limits will provide automatic code conformance with the cover and spacing requirements.

Table 2.8: Minimum number of bars in a single layer. (ACI 318-11, section 10.6)

Bar Size	Beam Width (in.)												
	12	14	16	18	20	22	24	26	28	30	36	42	48
No.4	2	2	3	3	3	3	3	4	4	4	5	5	6
No.5	2	2	3	3	3	3	3	4	4	4	5	5	6
No.6	2	2	3	3	3	3	3	4	4	4	5	5	6
No.7	2	2	3	3	3	3	3	4	4	4	5	5	6
No.8	2	2	3	3	3	3	3	4	4	4	5	5	6
No.9	2	2	3	3	3	3	3	4	4	4	5	5	6
No.10	2	2	3	3	3	3	3	4	4	4	5	5	6
No.11	2	2	3	3	3	3	3	4	4	4	5	5	6

The values in Table 2.8 are based on a cover of 2 in. to the main flexural reinforcement (i.e., 1.5 in. clear cover to the stirrups plus the diameter of a No. 4 stirrup). In general, the following equations can be used to determine the minimum number of bars n in a single layer for any situation.

$$n_{min} = \frac{b_w - 2(C_c + 0.5d_b)}{s} + 1$$

Where, $s = 15\times(40{,}000/f_s) - 2.5\times Cc \leq 12\times(4{,}000/f_s)$ and $f_s = 2/3\times f_y$

Table 2.9: Maximum number of bars in a single layer.

Bar Size	Beam Width (in)												
	12	14	16	18	20	22	24	26	28	30	36	42	48
No.4	5	6	8	9	10	12	13	14	16	17	21	25	29
No.5	5	6	7	8	10	11	12	13	15	16	19	23	27
No.6	4	6	7	8	9	10	11	12	14	15	18	22	25
No.7	4	5	6	7	8	9	10	11	12	13	17	20	23
No.8	4	5	6	7	8	9	10	11	12	13	16	19	22
No.9	3	4	5	6	7	8	8	9	10	11	14	17	19
No.10	3	4	4	5	6	7	8	8	9	10	12	15	17
No.11	3	3	4	5	5	6	7	8	8	9	11	13	15

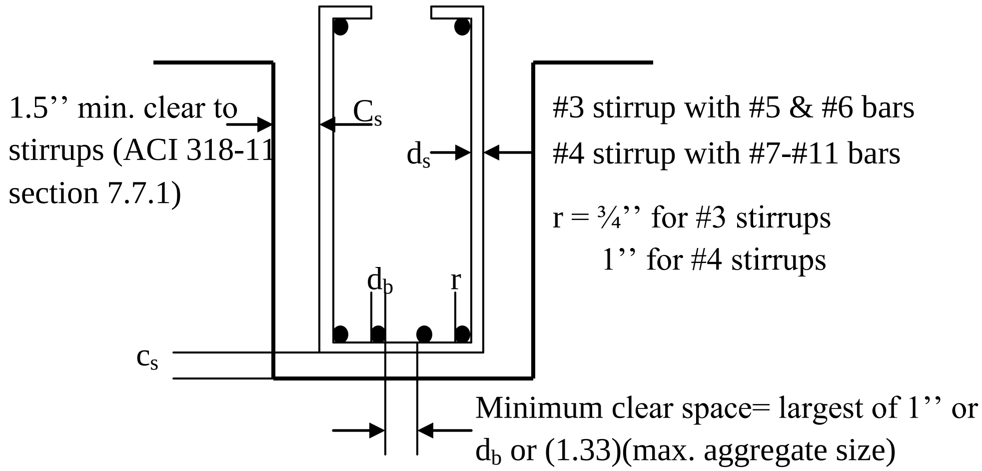


Figure 2.7: Cover and spacing requirements for Tables 2.8 and 2.9.

Where, b_w = beam width, in.

c_c = clear cover to tension reinforcement, in.

c_s = clear cover to stirrups, in.

d_b = diameter of main flexural bar, in.

d_s = diameter of stirrups.

The values obtained from the above equations should be rounded up to the next whole number. The values in Table 2.9 can be determined from the following equation:

$$n_{max} = 1 + \frac{b_w - 2(C_s + d_s + r)}{(\text{minimum clear space}) + d_b} \qquad \text{Where, } r = \begin{cases} \frac{3}{4}\text{ in. for No. 3 stirrups} \\ 1\text{ in. for No. 4 stirrups} \end{cases}$$

The minimum clear space between bars is defined in Figure 2.7. The above equation can be used to determine the maximum number of bars in any general case; computed values should be rounded down to the next whole number.

2.3.3 Selection of stirrups for economy

Selection of stirrup size and spacing for overall cost savings requires consideration of both design time and fabrication and placing costs.

Minimum cost solutions for simple placing should be limited to three spacing: the first stirrup located at two inch from the face of the support (as a minimum clearance), an intermediate spacing, and finally, a maximum spacing at the code limit of d/2. Larger size stirrups at wider spacings are more cost-effective (e.g., using No.4 and No.3 at double spacing and No.5 and No.4 at 1.5 spacing). If it is possible to use them within the spacing limitations of d/2 and d/4.

In order to adequately develop the stirrups, the following requirements must all be satisfied (ACI Code 318-11, section 12.13):

1) Stirrups shall be carried as close to the compression and tension surfaces of the member as cover requirements permit.

2) For No.5 stirrups and smaller, a standard stirrup hook (as defined in ACI Code 318-11, section 7.1.3) shall be provided around longitudinal reinforcement.

3) Each bend in the continuous portion of the stirrup must enclose a longitudinal bar. To allow for bend radii at corners of U stirrups, the minimum beam widths given in Table 2.10 should be provided.

Table 2.10: Minimum beam widths for stirrups.

Stirrup size	**Minimum beam width (b_w)**
No.3	10 in
No.4	12 in
No.5	14 in

2.3.4 Simplified design of beam stirrup

The design values in Table 2.11 are valid for $f'_c = 4000$ psi.

Table 2.11: Concrete shear strength design values for $f'_c = 4000$ psi.

Equation	Design Value	ACI 318-11 Section
$\varphi V_n = \varphi 2\sqrt{f'_c} b_w d$	$0.095 b_w d$	ACI 11.2.1.1
$0.5\varphi V_n = \varphi\sqrt{f'_c} b_w d$	$0.048 b_w d$	ACI 11.4.6.1
Maximum $\varphi V_c + \varphi V_n = \varphi 10\sqrt{f'_c} b_w d$	$0.48 b_w d$	ACI 11.4.7.9
Joists defined by ACI 8.13 $\varphi V_c = \varphi 2.2\sqrt{f'_c} b_w d$	$0.104 b_w d$	ACI 8.13.8

b_w and d are in inches and the resulting shear in kips.

The selection and spacing of stirrups can be simplified if the spacing is expressed as a function of the effective depth d. According to ACI 11.4.5.1 and ACI 11.4.5.3, the practical limits of stirrups spacing vary from s= d/2 to s= d/4 is not economical. With one intermediate spacing and d/3, the calculation and selection of stirrup spacing is greatly simplified. Using the three standard stirrup spacings noted above (d/2, d/3, d/4), a specific value of φV_s can be derived for each stirrup size and spacing as follows:

For vertical stirrups: $\varphi V_s = \frac{\varphi A_s f_y d}{s}$ …….(ACI equation 11.15)

By substituting d/n for s (where n= 2, 3, 4), the above equation can be written as:

$$\varphi V_s = \varphi A_s f_y n$$

Thus, for No 3 U-stirrups @ s= d/2 with $f_y = 60000$ psi and $\varphi = 0.75$

$\varphi V_s = 0.75(0.22)\times 60\times 2 = 19.8$ kips, say 20 kips

The values φV_s given in Table 2.12 may be used to select shear reinforcement with Grade 60 rebars.

Table 2.12: Values of φV_s (f_y= 60 ksi).

s	#3 U-stirrups	#4 U-stirrups	#5 U-stirrups
d/2	20 kips	36 kips	56 kips
d/3	30 kips	54 kips	84 kips
d/4	40 kips	72 kips	112 kips

*Valid for stirrups with 2 legs (double the tabulated values for 4 legs, etc).

It should be noted that the values of φV_s are not dependent on the member size nor on the concrete strength.

In the above equation b_w and d are in inches and the resulting shear in kips.

2.4 Development length

2.4.1 Cranked bar for slab

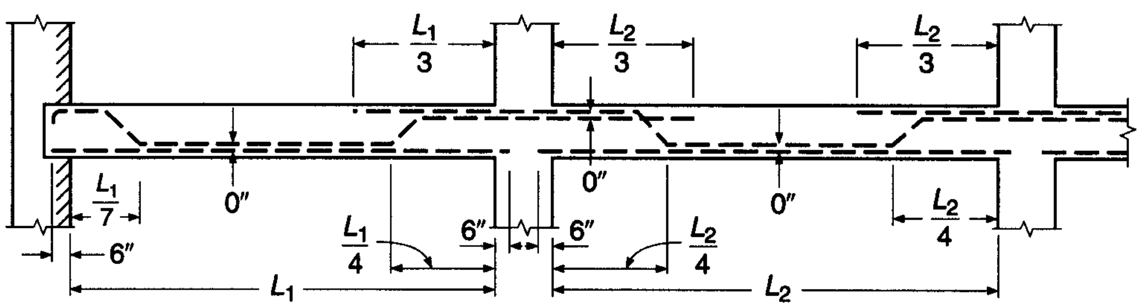


Figure 2.8: Rebar bend points in approximately equal spans with uniformly distributed loads.

2.4.2 Bar cutoff for beam

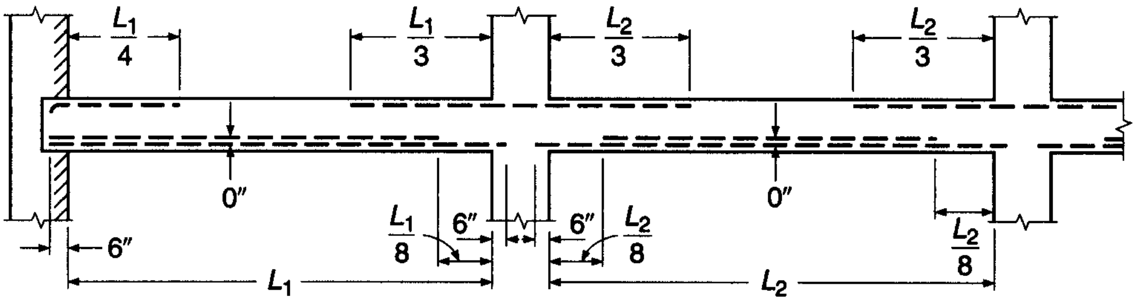


Figure 2.9: Rebar cutoff points in approximately equal spans with uniformly distributed loads.

2.5 Least squares (R^2)

The method of least squares is a standard approach in regression analysis to the approximate solution of over determined systems, i.e., sets of equations in which there are more equations than unknowns. "Least squares" means that the overall solution minimizes the sum of the squares of the errors made in the results of every single equation.

The most important application is in data fitting. The best fit in the least-squares sense minimizes the sum of squared residuals, a residual being the difference between an observed value and the fitted value provided by a model. When the problem has substantial uncertainties in the independent variable (the *x* variable), then simple regression and least squares methods have problems; in such cases, the methodology required for fitting errors-in-variables models may be considered instead of that for least squares.

Least squares problems fall into two categories: linear or ordinary least squares and non-linear least squares, depending on whether or not the residuals are linear in all unknowns.

The linear least-squares problem occurs in statistical regression analysis; it has a closed-form solution. The non-linear problem is usually solved by iterative refinement; at each iteration the system is approximated by a linear one, and thus the core calculation is similar in both cases.

Polynomial least squares describes the variance in a prediction of the dependent variable as a function of the independent variable and the deviations from the fitted curve.

When the observations come from an exponential family and mild conditions are satisfied, least-squares estimates and maximum-likelihood estimates are identical. The method of least squares can also be derived as a method of moments estimator.

2.6 Comparison table

2.6.1 Comparison table of design live loads for various occupancies suggested by different codes

A comparison table which contains design live loads for various occupancies suggested by different codes (i.e. ASCE, BNBC, IBC, UBC, Euro/ British Code) is given below:

Table 2.13: Table of design live loads for various occupancies suggested by different codes.

Occupancy or use	ASCE (2007) (KN/m^2)	BNBC (2006) (KN/m^2)	Euro Code (1991) (KN/m^2)	IBC (2012) (KN/m^2)	UBC (1997) (KN/m^2)
Residential					
• Uninhabitable attics without storages	0.48			0.48	
• Uninhabitable attics with storages	0.96			0.96	
• Habitable attics and sleeping areas	1.44	2.0	1.5 to 2.0	1.92	1.44
• All other areas except balconies and corridors	1.92	4.0	1.5 to 2.0	1.92	1.92
Balconies (exterior)	4.79	4.0	2.5 to 4	4.0	2.87
Stairs	2.87 to 4.79	2.0 to 3.0	2.0 to 4.0	1.92 (one or two family) 4.79 (for all other)	

Schools • Classrooms	1.92	3.0	2.0 to 3.0	1.92	1.92
Offices	2.4	2.5	2.0 to 3.0	2.4	2.4
Assembly Areas and theatres • With fixed seats • Without fixed seats	2.87 4.79	3.0 to 4.0 2.0 to 3.0	3.0 5.0	2.87 4.79	2.4 4.79
Hospitals • Operating rooms • Laboratories • wards	2.87 2.87 1.92	2.5 3.0 2.0	3.0 to 5.0 3.0 to 5.0 3.0 to 5.0	2.87 2.87 1.92	2.87 2.87 1.92
Stores	4.79	4.0	4.0 to 5.0	4.79	4.79
Storages • light • heavy	6.0 11.97	6.0 12.0	4.0 to 5.0 4.0 to 5.0	6.0 11.97	6.0 11.97
Manufacturing • light • heavy	6.0 11.97	6.0 12.0		6.0 11.97	6.0 11.97

Chapter 03

METHODOLOGY AND EXPERIMENTAL WORK

3.1 Two way slab

3.1.1 Simplified method of determination of thickness

Step 01: At first, we collect some full structural design and plan of building and then select 10 plans of floor slab among them for analysis. Those plans are given in the Appendix (Figure A.01- A.10).

Step 02: From each of the floor slab plan, we selected the largest slab panel for calculation of alpha.

Step 03: After that we take the dimensions of selected panel from each floor slab plan.

	Clear Span	
Plan No.	**Shorter length (in)**	**Longer Length (in)**
1	232	251
2	185	324
3	230	274
4	152	191
5	232	251
6	151	206
7	192	271
8	118	204
9	118	216.5
10	185.5	236

Step 04: Then we calculate alpha (α) for each selected slab panel of the floor slab plan according to Article 2.2.2.

Calculation of alpha for Plan 01 (Appendix Figure A.01):

α_1	**Beam Type**	α_2	**Beam Type**	α_3	**Beam Type**	α_4	**Beam Type**	**Avg. α**
4.03	Edge	2.57	Interior	4.33	Edge	3.61	Interior	3.64

Calculation of alpha for Plan 02 (Appendix Figure A.02):

α_1	**Beam Type**	α_2	**Beam Type**	α_3	**Beam Type**	α_4	**Beam Type**	**Avg. α**
4.53	Interior	6.47	Edge	6.84	Interior	10.86	Edge	7.18

Calculation of alpha for Plan 03 (Appendix Figure A.03):

α_1	**Beam Type**	α_2	**Beam Type**	α_3	**Beam Type**	α_4	**Beam Type**	**Avg. α**
3.28	Interior	4.58	Edge	5.39	Edge	5.39	Edge	4.66

Calculation of alpha for Plan 04 (Appendix Figure A.04):

α_1	**Beam Type**	α_2	**Beam Type**	α_3	**Beam Type**	α_4	**Beam Type**	**Avg. α**
3.72	Interior	5.17	Edge	6.34	Edge	3.84	Interior	4.77

Calculation of alpha for Plan 05 (Appendix Figure A.05):

α_1	**Beam Type**	α_2	**Beam Type**	α_3	**Beam Type**	α_4	**Beam Type**	**Avg. α**
4.03	Edge	0.24	Edge	4.33	Edge	4.33	Edge	3.23

Calculation of alpha for Plan 06 (Appendix Figure A.06):

α_1	**Beam Type**	**α_2**	**Beam Type**	**α_3**	**Beam Type**	**α_4**	**Beam Type**	**Avg. α**
4.83	Edge	3.5	Interior	6.38	Edge	4.17	Interior	4.72

Calculation of alpha for Plan 07 (Appendix Figure A.07):

α_1	**Beam Type**	**α_2**	**Beam Type**	**α_3**	**Beam Type**	**α_4**	**Beam Type**	**Avg. α**
3.75	Edge	2.87	Interior	3.7	Interior	5.15	Edge	3.87

Calculation of alpha for Plan 08 (Appendix Figure A.08):

α_1	**Beam Type**	**α_2**	**Beam Type**	**α_3**	**Beam Type**	**α_4**	**Beam Type**	**Avg. α**
5.26	Interior	4.4	Interior	4.87	Edge	4.87	Edge	4.85

Calculation of alpha for Plan 09 (Appendix Figure A.09):

α_1	**Beam Type**	**α_2**	**Beam Type**	**α_3**	**Beam Type**	**α_4**	**Beam Type**	**Avg. α**
4.87	Interior	4.28	Interior	4.61	Edge	4.61	Edge	4.59

Calculation of alpha for Plan 10 (Appendix Figure A.10):

α_1	**Beam Type**	**α_2**	**Beam Type**	**α_3**	**Beam Type**	**α_4**	**Beam Type**	**Avg. α**
7.42	Edge	5.46	Interior	7	Interior	9.24	Edge	7.28

Step 05: Determine minimum thickness of floor slab for each plan corresponding to alpha according to Article 2.2.1.

We can see that, for all floor slabs of the plan values of alpha, $\alpha_{avg} > 2$. So according to Article 2.2.1 we use ACI Eq. 9-13 for determining minimum slab thickness.

For α_m greater than 2.0, the thickness must not be less than

$$h= \frac{l_n (0.8+\left(\frac{fy}{200,000}\right))}{36+9\beta}$$ and not less than 3.5 inch….. (2.2) (ACI Eq. 9-13)

Where l_n = clear span in long direction, in.

α_m = average value of α for all beams on edges of a panel. [$\alpha = \frac{\mathbf{E_{cb}\ I_b}}{\mathbf{E_{cs}\ I_s}}$]

β = ratio of clear span in long direction to clear span in short direction.

and f_y = 60,000 psi.

	Clear Span			
Plan No.	**Shorter length (in)**	**Longer Length (in)**	**β**	**Thickness corresponding α (in)**
1	232	251	1.08	6.04
2	185	324	1.75	6.89
3	230	274	1.91	6.45
4	152	191	1.26	4.44
5	232	251	1.08	6.04
6	151	206	1.36	4.69
7	192	271	1.41	6.12
8	118	204	1.73	4.35
9	118	216.5	1.83	4.54
10	185.5	236	1.27	5.47

Step 06: Then we calculate perimeter (Inner, C/C, and outer) of the slab panel for all floor slab plan.

Plan no.	Inner perimeter (in)	C/C perimeter (in)	Outer perimeter (in)
01	966	1006	1046
02	1018	1058	1098
03	1008	1048	1088
04	686	726	766
05	966	1006	1046
06	714	754	794
07	926	966	1006
08	644	684	724
09	669	709	749
10	843	883	923

Step 07: Now, calculate thicknesses of slabs for floor slab plans from: $\frac{P_{Inner}}{145}$, $\frac{Pc/c}{145}$, $\frac{P_{Outer}}{145}$, $\frac{P}{150}$, $\frac{P}{160}$, and $\frac{P}{180}$ and compare them with the thicknesses comes from step 05.

Plan No.	Thickness corresponding α, (in)	$\frac{P_{Inner}}{145}$ (in)	$\frac{Pc/c}{145}$ (in)	$\frac{P_{Outer}}{145}$ (in)	$\frac{P}{150}$ (in)	$\frac{P}{160}$ (in)	$\frac{P}{180}$ (in)
01	6.04	6.66	6.94	7.21	6.97	6.54	5.81
02	6.89	7.02	7.3	7.57	7.32	6.86	6.1
03	6.45	6.95	7.23	7.5	7.25	6.8	6.04
04	4.44	4.73	5.01	5.28	5.11	4.79	4.26
05	6.04	6.66	6.94	7.21	6.97	6.54	5.81
06	4.69	4.92	5.2	5.48	5.29	4.96	4.41

Plan No.	Thickness corresponding α, (in)	$\frac{P_{Inner}}{145}$ (in)	$\frac{Pc/c}{145}$ (in)	$\frac{P_{Outer}}{145}$ (in)	$\frac{P}{150}$ (in)	$\frac{P}{160}$ (in)	$\frac{P}{180}$ (in)
07	6.12	6.39	6.66	6.94	6.71	6.29	5.59
08	4.35	4.44	4.72	4.99	4.83	4.53	4.02
09	4.54	4.61	4.89	5.17	4.99	4.68	4.16
10	5.47	5.81	6.09	6.37	6.15	5.77	5.13

Step 08: After that, we developed comparison graphs of: $\frac{P_{Inner}}{145}$ (in) vs Thickness corresponding α, (in); $\frac{Pc/c}{145}$ (in) vs Thickness corresponding α, (in); $\frac{P_{Outer}}{145}$ (in) vs Thickness corresponding α, (in); $\frac{P}{150}$ (in) vs Thickness corresponding α, (in); $\frac{P}{160}$ (in) vs Thickness corresponding α, (in); and $\frac{P}{180}$ (in) vs Thickness corresponding α, (in). For those 10 plans of floor slab.

Finally we found the "Least Squares (R^2)" value for each trend line of each graphs of calculated 10 plans of floor slab.

From the R^2 values of graphs, we can select the simplified thumb rule for determining thickness of slab.

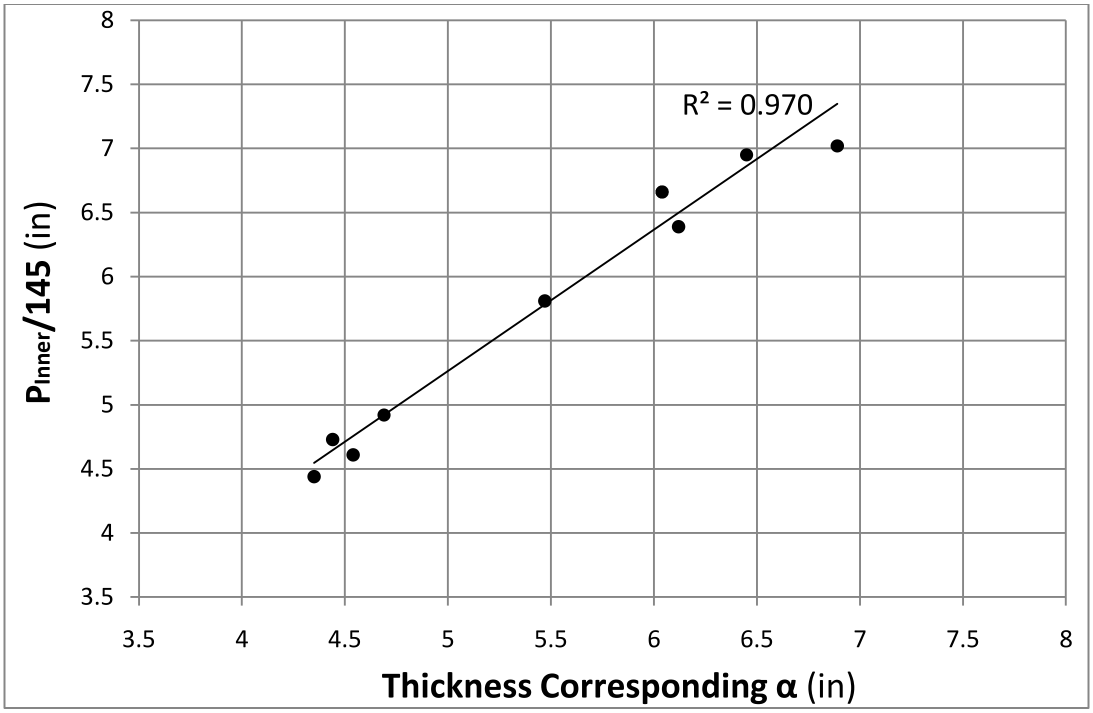


Figure 3.1: $\frac{P_{Inner}}{145}$ (in) vs Thickness corresponding α, (in) diagram.

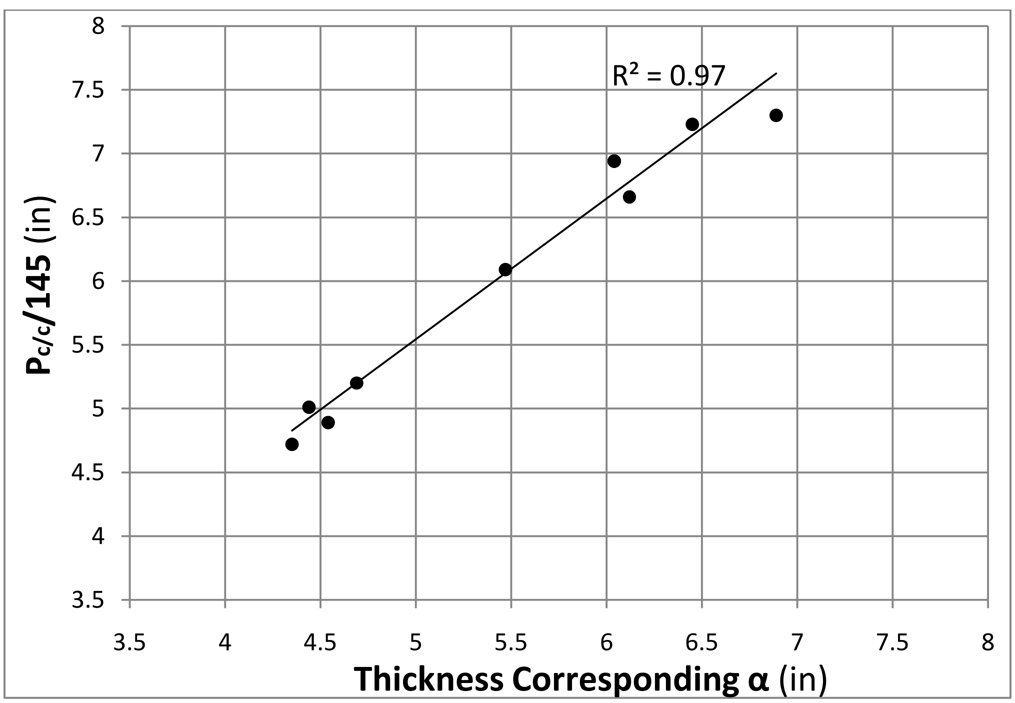


Figure 3.2: $\frac{Pc/c}{145}$ (in) vs Thickness corresponding α, (in) diagram.

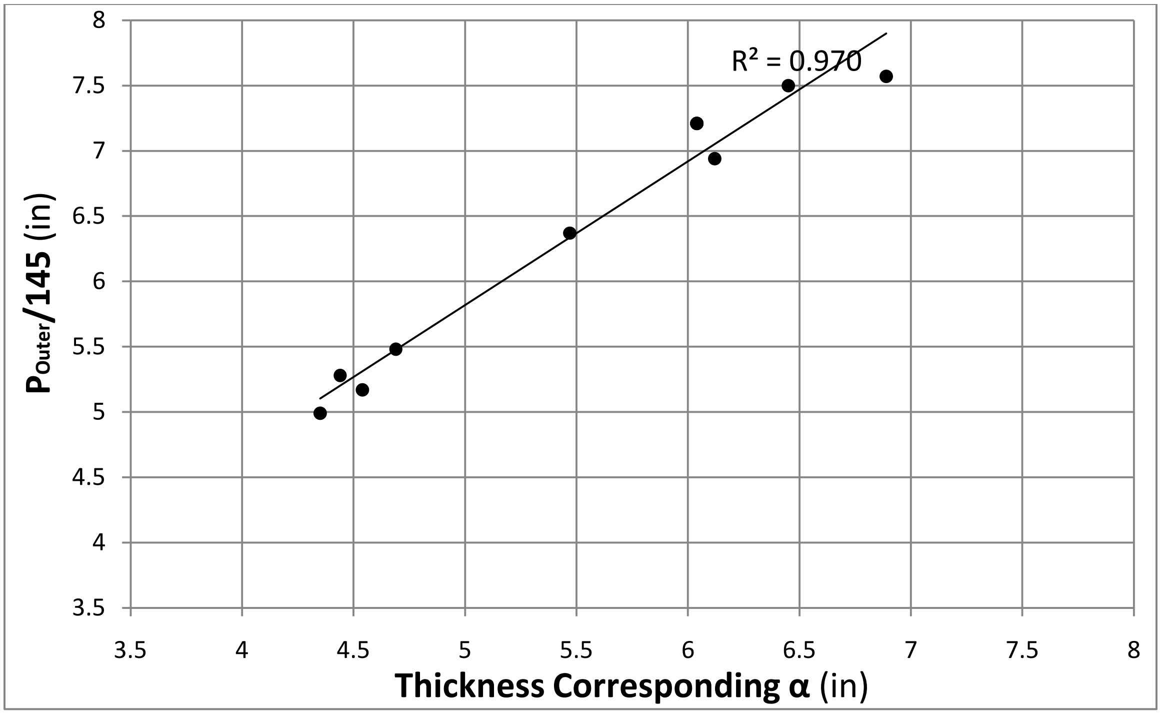


Figure 3.3: $\frac{P_{Outer}}{145}$ (in) vs Thickness corresponding α, (in) diagram.

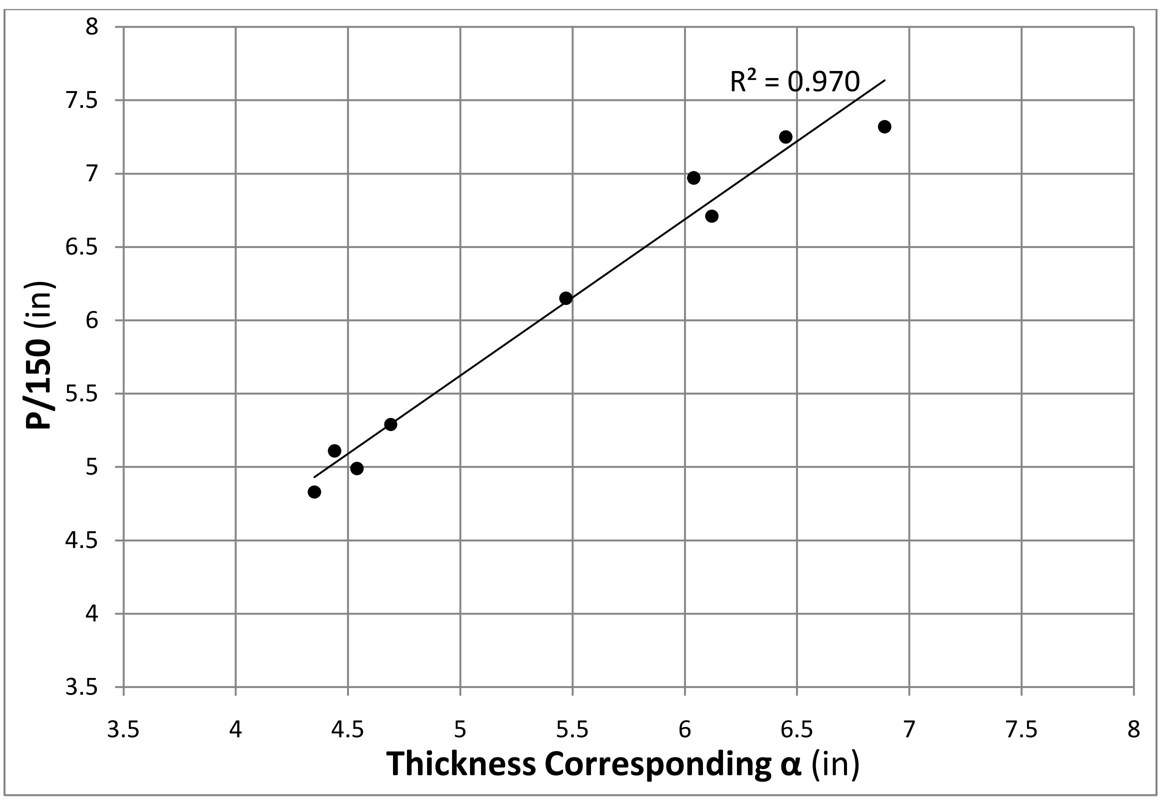


Figure 3.4: $\frac{P}{150}$ (in) vs Thickness corresponding α, (in) diagram.

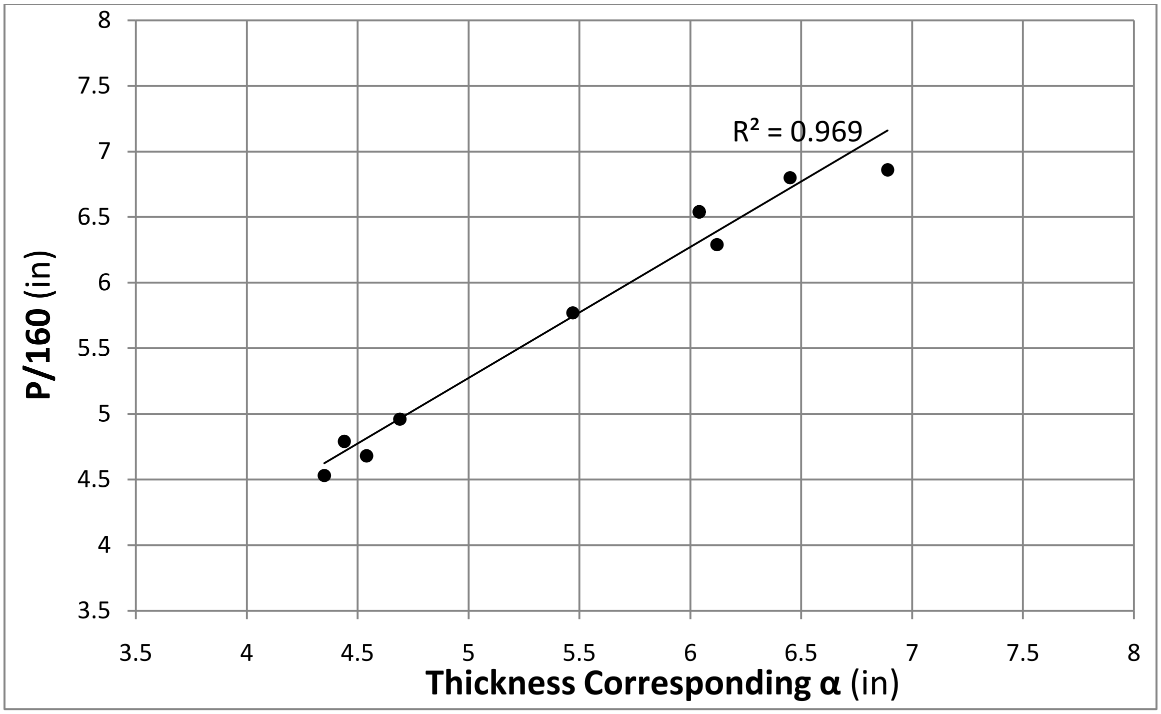


Figure 3.5: $\frac{P}{160}$ (in) vs Thickness corresponding α, (in) diagram.

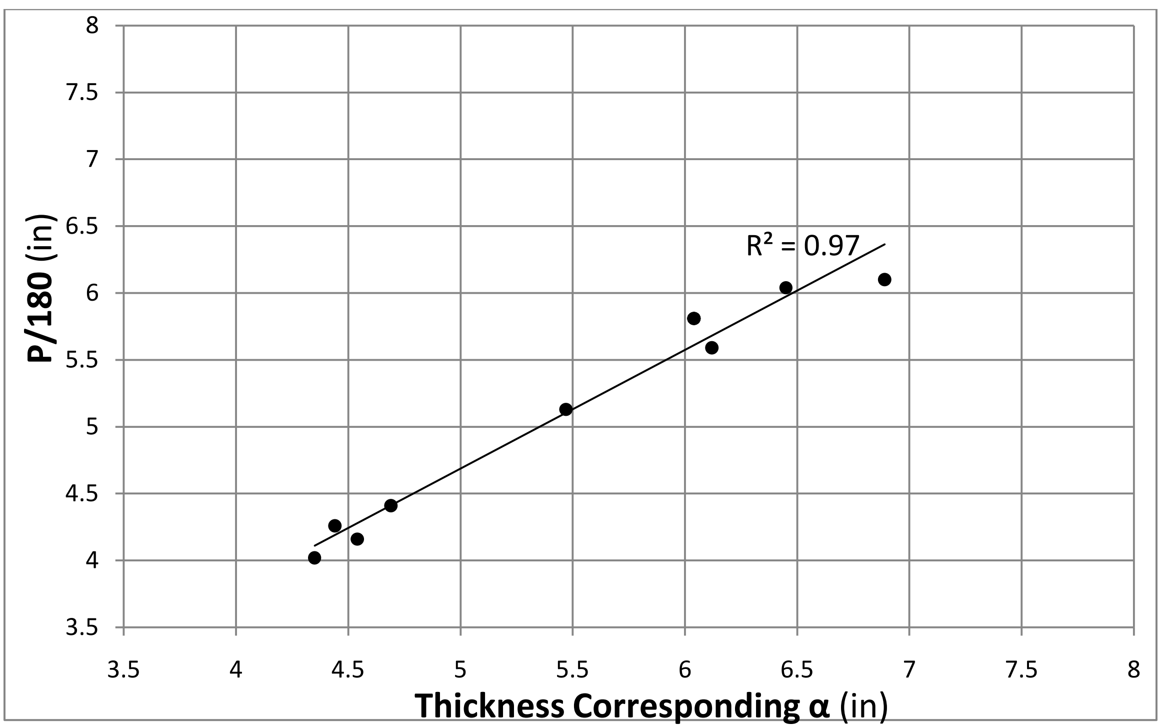


Figure 3.6: $\frac{P}{180}$ (in) vs Thickness corresponding α, (in) diagram.

Example #01:

Determination of minimum thickness of a slab

A two-way reinforced concrete building floor system is composed of slab panels measuring 20x25 ft in plan, supported by shallow column-line beams cast monolithically with the slab as shown in Figure 3.7. Using concrete with f'_c= 4000 psi and steel with f_y= 60,000 psi, determine the minimum thickness of the slab.

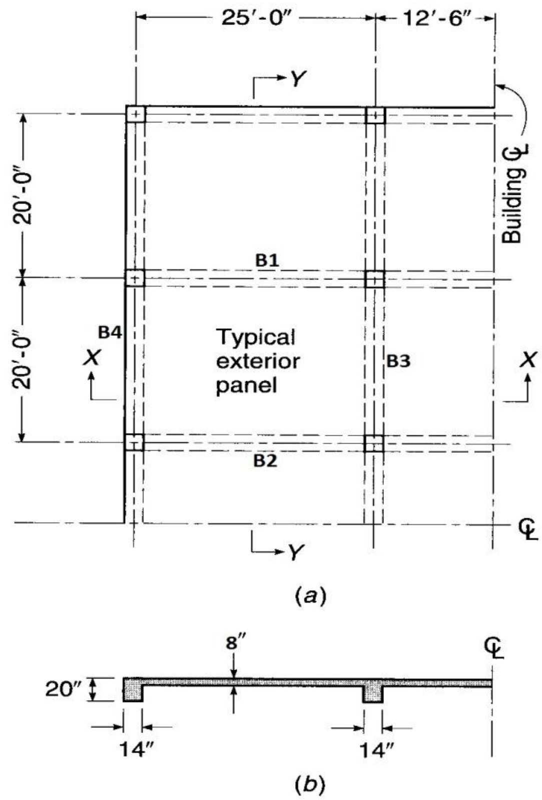


Figure 3.7: Two-way slab floor with beams on column lines:

(a) Partial floor plan;

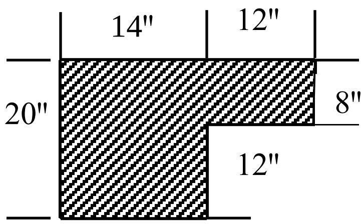
(b) Section *X*-*X* (section *Y*-*Y* similar).

Solution: At first select the largest slab panel from floor slab plan. In this example, dimension of the slab panel is 20'×25'. Primarily, now we determining thickness of slab using the following formula: thickness (in) = $\frac{\text{Perimeter}}{145}$

Here, Perimeter = 2×(20+25)×12= 1080 in

So thickness= $\frac{1080}{145}$ = 7.45 in≈ 8 in (say)

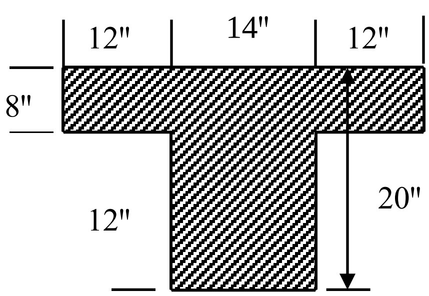
<u>Moment of Inertia for beam B4 (Exterior beam):</u>



$\bar{Y}_4 = \frac{(20\times14\times10)+(12\times8\times4)}{(20\times14)+(12\times8)}$= 8.47 in

$I_{b4} = \frac{14\times20^3}{12} + (20\times14)\times(10-8.47)^2 + \frac{12\times8^3}{12} + (12\times8)\times(8.47-4)^2 =$ 12418.95 in^4

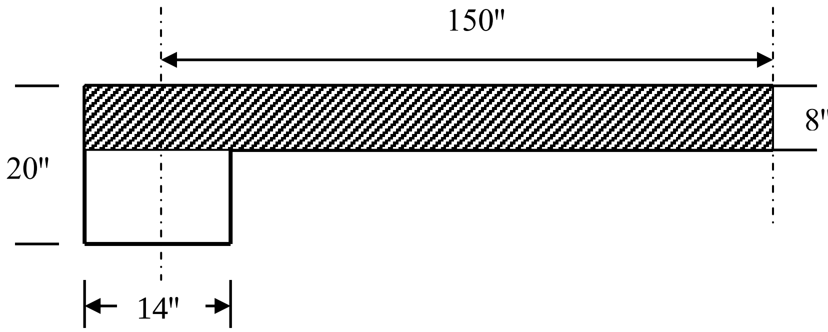
<u>Moment of Inertia for Beam B1, B2, and B3 (Interior beam):</u> In this case, those three beam's dimension is same and those are interior beam. So here moment of inertia for beam B1, B2, B3 is same.



$\bar{Y}_1 = \bar{Y}_2 = \bar{Y}_3 = \frac{(20\times14\times10)+(12\times8\times4)\times2}{(20\times14)+(12\times8)\times2}$= 7.56 in

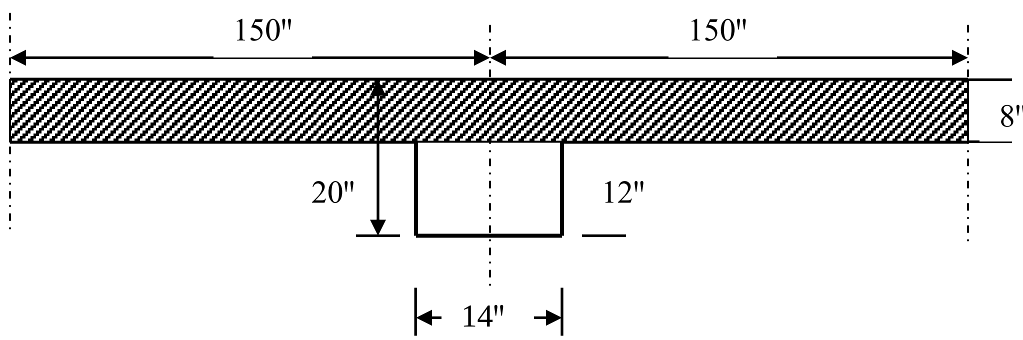
$I_{b1} = I_{b2} = I_{b3} = \frac{14\times20^3}{12} + (20\times14)\times(10-7.56)^2 + (\frac{12\times8^3}{12} + (12\times8)\times(7.56-4)^2)\times 2$= 14457.67 in^4

Calculation of I_{s4}:



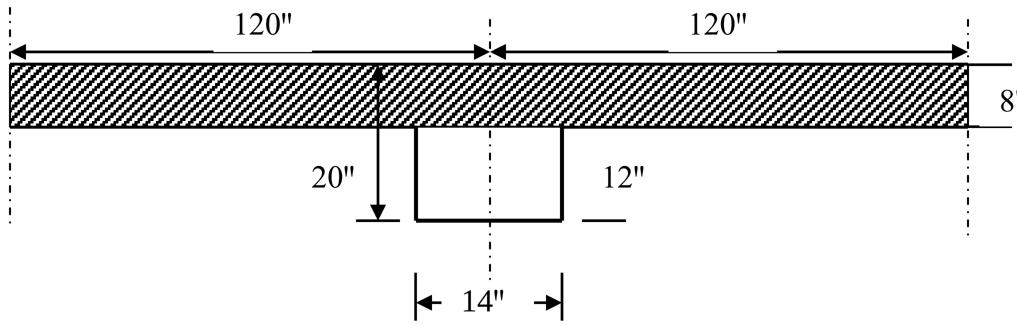
$I_{s4} = \frac{(150+7)\times 8^3}{12} = 6698.67 \text{ in}^4$

Calculation of I_{s3}:



$I_{s3} = \frac{(150+150)\times 8^3}{12} = 12800 \text{ in}^4$

Calculation of I_{s1} and I_{s2}: Value of I_{s1} and I_{s2} is same. Because B1 and B2 both are interior beam and for both cases, clear span on both side transverse to the beam B1 and B2 are same.



$I_{s1} = I_{s2} = \frac{(120+120)\times 8^3}{12} = 10240$ in^4

<u>Calculation of α:</u> We know $\alpha = \frac{\mathbf{E_{cb}\,I_b}}{\mathbf{E_{cs}\,I_s}}$. Here $E_{cb} = E_{cs}$. Because of beam and slab concrete is same. So we can write $\alpha = \frac{\mathbf{I_b}}{\mathbf{I_s}}$.

For this example $\alpha_1 = \frac{\mathbf{I_{b1}}}{\mathbf{I_{s1}}} = \frac{14457.67}{10240} = 1.41$

$\alpha_2 = \frac{\mathbf{I_{b2}}}{\mathbf{I_{s2}}} = \frac{14457.67}{10240} = 1.41$

$\alpha_3 = \frac{\mathbf{I_{b3}}}{\mathbf{I_{s3}}} = \frac{14457.67}{12800} = 1.13$

$\alpha_4 = \frac{\mathbf{I_{b4}}}{\mathbf{I_{s4}}} = \frac{12418.95}{6698.67} = 1.85$

Average value of α, $\alpha_{avg} = \frac{1.41+1.41+1.13+1.85}{4} = 1.45$

The ratio of long to short clear spans is $\beta = {}^{286}/_{226} = 1.27$. Then the minimum thickness is not to be less than that given by Eq. (2.1) (Article 2.2.1):

$$h = \frac{286\left(0.8+\left(\frac{60000}{200{,}000}\right)\right)}{36+5\times 1.27(1.45-0.2)} = 7.16 \text{ in}$$

3.1.2 Design Procedure of Two-way Slabs using ACI Moment Coefficients:

The method makes use of tables of moment coefficient for a variety of conditions. These coefficients are based on elastic analysis but also account for inelastic redistribution. This method was recommended in 1963 ACI Code for the special case of two-way slabs supported on four sides by relatively deep, stiff, edge beams.

Step 01: Determine minimum thickness of the slab.
Determine the thickness of the slab, h using Article 2.2.1.

Step 02: Calculation of factored load.

W_{DL}= 1.2*DL and W_{LL}= 1.6*LL;

$W_u = W_{DL} + W_{LL}$

Where DL= Total dead load (i.e.: Slab self weight, Floor finish, Partition wall, Plaster etc.)

LL= Live load.

Step 03: Determination of moment coefficients.

$$m = \frac{A}{B}$$

Where A= Shorter length of the slab.

B= Longer length of the slab.

Case type is identified from end condition. Using the value of 'm' corresponding moment coefficients are obtained for respective 'case type' from Table 3.1, 3.2 and 3.3. The co-efficients are:

- $C_{A\ neg}$ and $C_{B\ neg}$
- $C_{A\ DL\ pos}$ and $C_{B\ DL\ pos}$
- $C_{A\ LL\ pos}$ and $C_{B\ LL\ pos}$

Table 3.1: Table for $C_{A\ neg}$ and $C_{B\ neg}$.[3.1]

Ratio $m = \frac{A}{B}$		Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
1.00	$C_{A\ neg}$		0.045		0.050	0.075	0.071		0.033	0.061
	$C_{B\ neg}$		0.045	0.076	0.050			0.071	0.061	0.033
0.95	$C_{A\ neg}$		0.050		0.055	0.079	0.075		0.038	0.065
	$C_{B\ neg}$		0.041	0.072	0.045			0.067	0.056	0.029
0.90	$C_{A\ neg}$		0.055		0.060	0.080	0.079		0.043	0.068
	$C_{B\ neg}$		0.037	0.070	0.040			0.062	0.052	0.025
0.85	$C_{A\ neg}$		0.060		0.066	0.082	0.083		0.049	0.072
	$C_{B\ neg}$		0.031	0.065	0.034			0.057	0.046	0.021
0.80	$C_{A\ neg}$		0.065		0.071	0.083	0.086		0.055	0.075
	$C_{B\ neg}$		0.027	0.061	0.029			0.051	0.041	0.017
0.75	$C_{A\ neg}$		0.069		0.076	0.085	0.088		0.061	0.078
	$C_{B\ neg}$		0.022	0.056	0.024			0.044	0.036	0.014
0.70	$C_{A\ neg}$		0.074		0.081	0.086	0.091		0.068	0.081
	$C_{B\ neg}$		0.017	0.050	0.019			0.038	0.029	0.011
0.65	$C_{A\ neg}$		0.077		0.085	0.087	0.093		0.074	0.083
	$C_{B\ neg}$		0.014	0.043	0.015			0.031	0.024	0.008
0.60	$C_{A\ neg}$		0.081		0.089	0.088	0.095		0.080	0.085
	$C_{B\ neg}$		0.010	0.035	0.011			0.024	0.018	0.006
0.55	$C_{A\ neg}$		0.084		0.092	0.089	0.096		0.085	0.086
	$C_{B\ neg}$		0.007	0.028	0.008			0.019	0.014	0.005
0.50	$C_{A\ neg}$		0.086		0.094	0.090	0.097		0.089	0.088
	$C_{B\ neg}$		0.006	0.022	0.006			0.014	0.010	0.003

*A cross-hatched edge indicates that the slab continues across or is fixed at the support; an unmarked edge indicates a support at which torsional resistance is negligible.

Table 3.2: Table for $C_{A\ DL\ pos}$ and $C_{B\ DL\ pos}$.[3.2]

Ratio $m = \frac{A}{B}$		Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
1.00	$C_{A\ DL}$	0.036	0.018	0.018	0.027	0.027	0.033	0.027	0.020	0.023
	$C_{B\ DL}$	0.036	0.018	0.027	0.027	0.018	0.027	0.033	0.023	0.020
0.95	$C_{A\ DL}$	0.040	0.020	0.021	0.030	0.028	0.036	0.031	0.022	0.024
	$C_{B\ DL}$	0.033	0.016	0.025	0.024	0.015	0.024	0.031	0.021	0.017
0.90	$C_{A\ DL}$	0.045	0.022	0.025	0.033	0.029	0.039	0.035	0.025	0.026
	$C_{B\ DL}$	0.029	0.014	0.024	0.022	0.013	0.021	0.028	0.019	0.015
0.85	$C_{A\ DL}$	0.050	0.024	0.029	0.036	0.031	0.042	0.040	0.029	0.028
	$C_{B\ DL}$	0.026	0.012	0.022	0.019	0.011	0.017	0.025	0.017	0.013
0.80	$C_{A\ DL}$	0.056	0.026	0.034	0.039	0.032	0.045	0.045	0.032	0.029
	$C_{B\ DL}$	0.023	0.011	0.020	0.016	0.009	0.015	0.022	0.015	0.010
0.75	$C_{A\ DL}$	0.061	0.028	0.040	0.043	0.033	0.048	0.051	0.036	0.031
	$C_{B\ DL}$	0.019	0.009	0.018	0.013	0.007	0.012	0.020	0.013	0.007
0.70	$C_{A\ DL}$	0.068	0.030	0.046	0.046	0.035	0.051	0.058	0.040	0.033
	$C_{B\ DL}$	0.016	0.007	0.016	0.011	0.005	0.009	0.017	0.011	0.006
0.65	$C_{A\ DL}$	0.074	0.032	0.054	0.050	0.036	0.054	0.065	0.044	0.034
	$C_{B\ DL}$	0.013	0.006	0.014	0.009	0.004	0.007	0.014	0.009	0.005
0.60	$C_{A\ DL}$	0.081	0.034	0.062	0.053	0.037	0.056	0.073	0.048	0.036
	$C_{B\ DL}$	0.010	0.004	0.011	0.007	0.003	0.006	0.012	0.007	0.004
0.55	$C_{A\ DL}$	0.088	0.035	0.071	0.056	0.038	0.058	0.081	0.052	0.037
	$C_{B\ DL}$	0.008	0.003	0.009	0.005	0.002	0.004	0.009	0.005	0.003
0.50	$C_{A\ DL}$	0.095	0.037	0.080	0.059	0.039	0.061	0.089	0.056	0.038
	$C_{B\ DL}$	0.006	0.002	0.007	0.004	0.001	0.003	0.007	0.004	0.002

*A cross-hatched edge indicates that the slab continues across or is fixed at the support; an unmarked edge indicates a support at which torsional resistance is negligible.

Table 3.3: Table for $C_{A\ LL\ pos}$ and $C_{B\ LL\ pos}$.[3.3]

Ratio $m = \frac{A}{B}$		Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
1.00	$C_{A\ LL}$	0.036	0.027	0.027	0.032	0.032	0.035	0.032	0.028	0.030
	$C_{B\ LL}$	0.036	0.027	0.032	0.032	0.027	0.032	0.035	0.030	0.028
0.95	$C_{A\ LL}$	0.040	0.030	0.031	0.035	0.034	0.038	0.036	0.031	0.032
	$C_{B\ LL}$	0.033	0.025	0.029	0.029	0.024	0.029	0.032	0.027	0.025
0.90	$C_{A\ LL}$	0.045	0.034	0.035	0.039	0.037	0.042	0.040	0.035	0.036
	$C_{B\ LL}$	0.029	0.022	0.027	0.026	0.021	0.025	0.029	0.024	0.022
0.85	$C_{A\ LL}$	0.050	0.037	0.040	0.043	0.041	0.046	0.045	0.040	0.039
	$C_{B\ LL}$	0.026	0.019	0.024	0.023	0.019	0.022	0.026	0.022	0.020
0.80	$C_{A\ LL}$	0.056	0.041	0.045	0.048	0.044	0.051	0.051	0.044	0.042
	$C_{B\ LL}$	0.023	0.017	0.022	0.020	0.016	0.019	0.023	0.019	0.017
0.75	$C_{A\ LL}$	0.061	0.045	0.051	0.052	0.047	0.055	0.056	0.049	0.046
	$C_{B\ LL}$	0.019	0.014	0.019	0.016	0.013	0.016	0.020	0.016	0.013
0.70	$C_{A\ LL}$	0.068	0.049	0.057	0.057	0.051	0.060	0.063	0.054	0.050
	$C_{B\ LL}$	0.016	0.012	0.016	0.014	0.011	0.013	0.017	0.014	0.011
0.65	$C_{A\ LL}$	0.074	0.053	0.064	0.062	0.055	0.064	0.070	0.059	0.054
	$C_{B\ LL}$	0.013	0.010	0.014	0.011	0.009	0.010	0.014	0.011	0.009
0.60	$C_{A\ LL}$	0.081	0.058	0.071	0.067	0.059	0.068	0.077	0.065	0.059
	$C_{B\ LL}$	0.010	0.007	0.011	0.009	0.007	0.008	0.011	0.009	0.007
0.55	$C_{A\ LL}$	0.088	0.062	0.080	0.072	0.063	0.073	0.085	0.070	0.063
	$C_{B\ LL}$	0.008	0.006	0.009	0.007	0.005	0.006	0.009	0.007	0.006
0.50	$C_{A\ LL}$	0.095	0.066	0.088	0.077	0.067	0.078	0.092	0.076	0.067
	$C_{B\ LL}$	0.006	0.004	0.007	0.005	0.004	0.005	0.007	0.005	0.004

*A cross-hatched edge indicates that the slab continues across or is fixed at the support; an unmarked edge indicates a support at which torsional resistance is negligible.

Step 04: Determine controlling moments $+M_A$, $-M_A$, $+M_B$, $-M_B$.

$+M_A= C_{A\ DL}\times W_{DL}\times A^2+ C_{A\ LL}\times W_{LL}\times A^2$;

$-M_A= C_{A\ neg}\times W_u\times A^2$;

$+M_B= C_{B\ DL}\times W_{DL}\times B^2+ C_{B\ LL}\times W_{LL}\times B^2$;

$-M_B= C_{B\ neg}\times W_u\times B^2$.

Step 05: Determine required steel area.

$A_s= \frac{M_u}{Ø*f_y*(d-\frac{a}{2})}$; Here Ø= 0.9

And $a= \frac{A_s*f_y}{0.85*f'c*b}$

According to ACI Code 318-11, section 13.3.1 the minimum reinforcement in each direction shall be as mentioned below:

For 40 grade rebar: $A_{s\ min}$= 0.0020×b×h

60 grade rebar: $A_{s\ min}$= 0.0018×b×h

$>$ 60 grade rebar: $A_{s\ min}= \frac{0.0018\times 60{,}000}{f_y}\times b\times h$

Step 06: Determine c/c spacing of used bars.

Using # 3 or # 4 bar required spacing is determined. Maximum spacing < 2h (ACI Code 318-11, section 13.3.2).

Example #02:

Design of two-way edge supported slab by using moment coefficients

Beam-column supported floor slab of a 93'-6"×75'-6" (center to center distance of extreme columns) cyclone shelter is to carry service live load of 100 psf in addition to its own weight, 1/2" thick plaster and 3/2" thick floor finish. Supporting columns of 14 in square are spaced orthogonally at an interval at 31'-2" and 25'-2" on centers along longitudinal and transverse directions respectively. Width of each beam is 14 in. Using BNBC/ACI code of moment coefficients design the slab by USD method, if f'_c= 3000 psi and f_y= 60000 psi.

Solution:

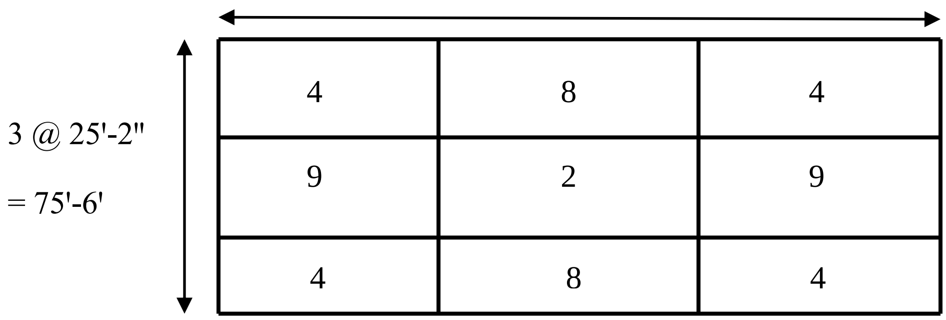


Figure 3.8: Slab panel orientation and case types.

Here, A= 25'2"-1'2"= 24' and B= 31'2"-1'2"= 30'= l_n.

t= $\frac{l_n(0.8+\left(\frac{fy}{200000}\right))}{36+9\beta} = \frac{30*(0.8+\left(\frac{60000}{200000}\right))}{36+9*\frac{30}{24}}$= 8.38"≈ 8.5" say.

So, d= 8.5"-1"= 7.5'

W_{DL}= (8.5+0.5+1.5)*12.5*1.2= 157.5 psf

W_{LL}= 100*1.6= 160 psf

W_u = 317.5 psf

m= A/B= 24/30= 0.8

	2	4	8	9
$-C_A$	0.065	0.071	0.055	0.075
$-C_B$	0.027.	0.029	0.041	0.017
$C_{A\ DL}$	0.026	0.039	0.032	0.029
$C_{B\ DL}$	0.011	0.016	0.015	0.010
$C_{A\ LL}$	0.041	0.048	0.044	0.042
$C_{B\ LL}$	0.017	0.020	0.019	0.017

Controlling coefficient.

Note 3.1: In this slab, there are four different types of cases among all panels. We take the maximum value of moment coefficient from four cases.

$+M_A = C_{A\,DL}*W_{DL}*A^2 + C_{A\,LL}*W_{LL}*A^2$

$= 0.039*157.5*24^2 + 0.048*160*24^2$

= 7961.76l lb-ft/ft

= 7.69 k-ft/ft

$-M_A = C_A*W_u*A^2$

$= 0.075*317.5*24^2$

= 13716 lb-ft/ft

= 13.6 k-ft/ft

$+M_B = C_{B\,DL}*W_{DL}*B^2 + C_{B\,LL}*W_{LL}*B^2$

$= 0.016*157.5*30^2 + 0.020*160*30^2$

= 5148 lb-ft/ft

= 5.148 k-ft/ft

$-M_B = C_B*W_u*B^2$

$= 0.041*317.5*30^2$

= 11716 lb-ft/ft

= 11.716 k-ft/ft

<u>Rebar for short direction/transverse direction:</u>

$+A_{S\,A} = \frac{M*12}{0.9*f_y*\left(d-\frac{a}{2}\right)} = \frac{M*12}{0.9*60*\left(d-\frac{a}{2}\right)} = \frac{M}{4.5*\left(d-\frac{a}{2}\right)} = \frac{7.96}{4.5*(7.5-0.24)} = 0.244\ in^2/ft$

and $a = \frac{A_s f_y}{0.85 f_c' b} = \frac{A_s*60}{0.85*3*12} = 1.96*A_S = 1.96*0.244 = 0.478$ in.

$A_{s\ min}$ = 0.0018*b*t*1.5 = 0.0018*12*8.5*1.5 = 0.275 in^2/ft (Controlling).

Using Φ 10 mm bar,

S = $\frac{\text{area of bar used} * \text{width of strip}}{\text{requried } A_s}$ = $\frac{0.121*12}{0.275}$ = 5.28" ≈ 5"c/c at bottom along short direction

crank 50% bar to negative zone.

-$A_{S\ A}$= $\frac{M}{4.5*\left(d-\frac{a}{2}\right)}$ = $\frac{13.61}{4.5*(7.5-0.42)}$ = 0.427 in^2/ft (Controlling).

$a = 1.96 * A_s$ = 0.838 in

$A_{s\ min}$ = 0.275 in^2/ft.

Already provided, A_{s1}= $\frac{0.121*12}{10}$ = 0.1452 in^2/ft

Extra top required, A_{s2}= (0.4275-0.1452) = 0.2823 in^2/ft.

Using Φ10 mm bar, S= 5.14" ≈ 5" c/c extra top.

<u>Rebar along long direction:</u>

+$A_{S\ B}$ = $\frac{5.148}{4.5*(7.5-0.15)}$ = 0.155 in^2/ft

$A_{s\ min}$ = 0.275 in^2/ft (Controlling).

Using Φ10 mm bar @ 5.27' '≈ 5" c/c at bottom along long direction crank 50% bar to negative zone.

-$A_{S\ B}$ = $\frac{11.716}{4.5*(7.5-0.36)}$ = 0.365 in^2/ft

Already provided, A_{s1}= $\frac{0.121*12}{10}$ = 0.145 in^2/ft

Extra top required, A_{s2}= (0.365-0.1452) in^2/ft = 0.2198 in^2/ft

Using Φ10 mm bar @ 6.6" ≈ 6.5" c/c extra top.

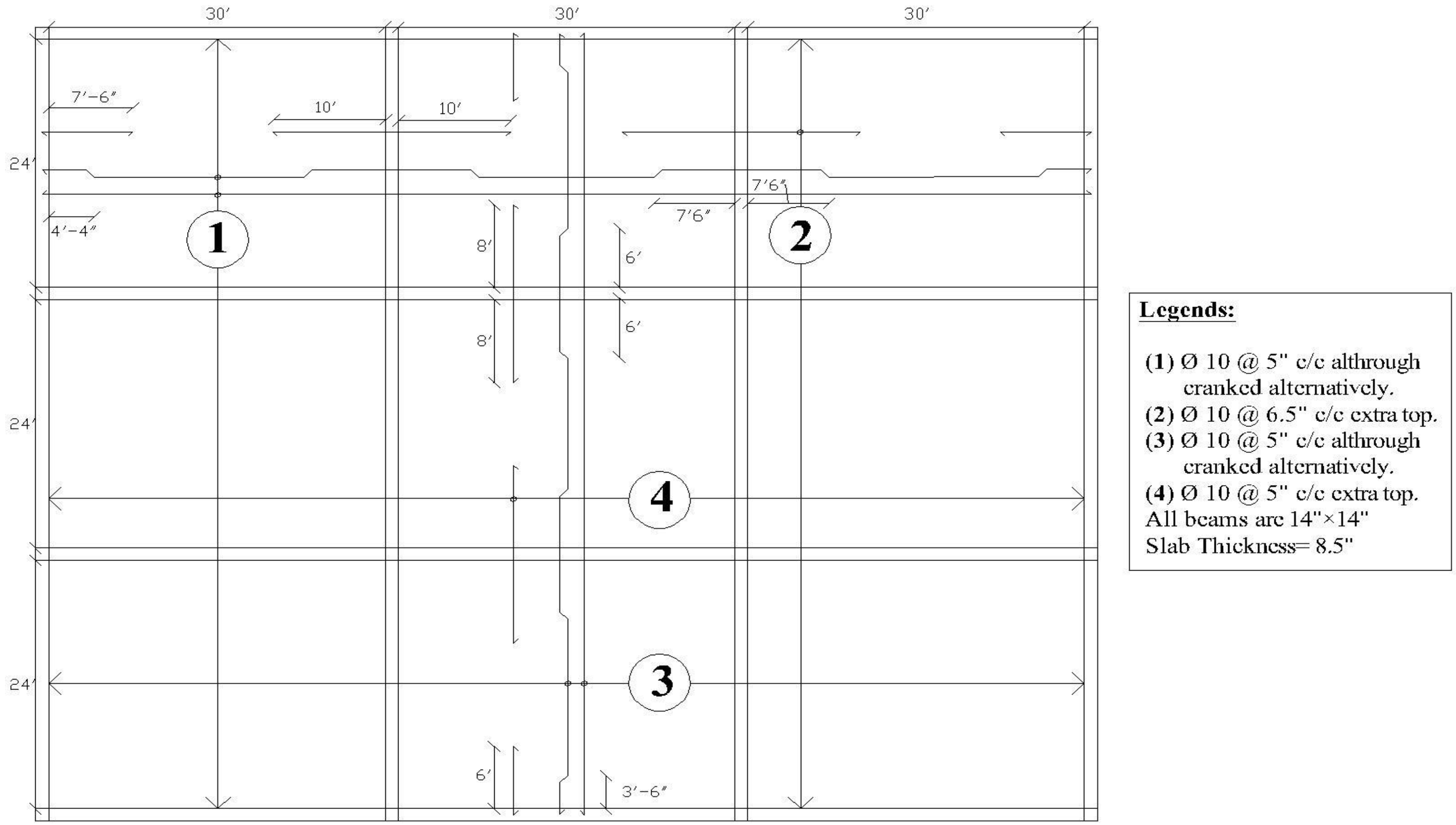


Figure 3.9: Reinforcement details of slab in plan (Example #02).

3.1.3 Relationship between length and steel area

Step 01: First of all, we select 13 slab panels randomly. Since, $\frac{\text{Shorter length}}{\text{Longer length}} \geq 0.5$ for all slab panels, so they are all two way slab.

Slab No.	Shorter length (in)	Longer length (in)
01	96	120
02	108	132
03	120	144
04	132	156
05	144	168
06	156	180
07	168	192
08	180	204
09	192	216
10	204	228
11	216	240
12	228	252
13	240	264

Step 02: Determine thickness for each slab panel by using: $\frac{\text{Perimeter}}{145}$ (in) and roundup them to the next 0.5 in.

Slab No.	Shorter length (in)	Longer length (in)	Thickness from $\frac{\text{Perimeter}}{145}$ (in)	Rounded thickness (in)
01	96	120	2.97931	3.5
02	108	132	3.310345	3.5
03	120	144	3.641379	4

Slab No.	Shorter length (in)	Longer length (in)	Thickness from $\frac{\text{Perimeter}}{145}$ (in)	Rounded thickness (in)
04	132	156	3.972414	4
05	144	168	4.303448	4.5
06	156	180	4.634483	5
07	168	192	4.965517	5
08	180	204	5.296552	5.5
09	192	216	5.627586	6
10	204	228	5.958621	6
11	216	240	6.289655	6.5
12	228	252	6.62069	7
13	240	264	6.951724	7

Step 03: Calculation of factored load.

W_{DL}= 1.2*DL and W_{LL}= 1.6*LL;

$W_T = W_{DL}+W_{LL}$

Where DL= Total dead load (i.e.: Slab self weight, Floor finish, Partition wall, Plaster etc.)

Note 3.2: In this case, DL is only slab self weight. Unit weight of concrete= 150 lb/ft^3

LL= Live load = 60 psf.

Slab No.	W_{DL} (ksf)	W_{LL} (ksf)	W_T (ksf)
01	0.045	0.096	0.141
02	0.0525	0.096	0.1485
03	0.06	0.096	0.156
04	0.06	0.096	0.156
05	0.0675	0.096	0.1635
06	0.075	0.096	0.171

07	0.075	0.096	0.171
08	0.0825	0.096	0.1785
09	0.09	0.096	0.186
10	0.09	0.096	0.186
11	0.0975	0.096	0.1935
12	0.105	0.096	0.201
13	0.105	0.096	0.201

Step 04: Taking the maximum values of moment coefficients from Table 3.1, 3.2 and 3.3.

Those are: $C_{A\ neg}^{max} = 0.097$

$C_{B\ neg}^{max} = 0.076$

$C_{A\ DL}^{max} = 0.095$

$C_{B\ DL}^{max} = 0.036$

$C_{A\ LL}^{max} = 0.095$

$C_{B\ LL}^{max} = 0.036$

Using those values calculating moments from those equations:

Controlling moments $+M_A$, $-M_A$, $+M_B$, $-M_B$.

$+M_A = C_{A\ DL}^{max} \times W_{DL} \times A^2 + C_{A\ LL}^{max} \times W_{LL} \times A^2$;

$-M_A = C_{A\ neg}^{max} \times W_u \times A^2$;

$+M_B = C_{B\ DL}^{max} \times W_{DL} \times B^2 + C_{B\ LL}^{max} \times W_{LL} \times B^2$;

$-M_B = C_{B\ neg}^{max} \times W_u \times B^2$.

Slab No.	+$\mathbf{M_A}$ (k-ft/ft)	-$\mathbf{M_A}$ (k-ft/ft)	+$\mathbf{M_B}$ (k-ft/ft)	-$\mathbf{M_B}$ (k-ft/ft)
01	0.85728	0.875328	0.5076	1.0716
02	1.142708	1.166765	0.646866	1.365606
03	1.482	1.5132	0.808704	1.707264
04	1.79322	1.830972	0.949104	2.003664
05	2.23668	2.283768	1.153656	2.435496
06	2.745405	2.803203	1.3851	2.9241
07	3.18402	3.251052	1.575936	3.326976
08	3.815438	3.895763	1.857114	3.920574
09	4.52352	4.618752	2.169504	4.580064
10	5.10663	5.214138	2.417256	5.103096
11	5.95593	6.081318	2.7864	5.8824
12	6.893295	7.038417	3.191076	6.736716
13	7.638	7.7988	3.502224	7.393584

Step 05: Determine required steel area.

$A_s = \frac{M_u}{Ø * f_y * (d - \frac{a}{2})}$; Here, Ø= 0.9

And $a = \frac{A_s * f_y}{0.85 * f'c * b}$

Note 3.3: Taking initially a= 1 for first trial.

According to ACI Code 318-11, section 13.3.1 the minimum reinforcement in each direction shall be as mentioned below:

For 40 grade rebar: $A_{s\ min}$= 0.0020×b×h

60 grade rebar: $A_{s\ min}$= 0.0018×b×h

> 60 grade rebar: $A_{s\ min} = \frac{0.0018 \times 60{,}000}{f_y} \times b \times h$

Note 3.4: In this case we take, f_y = 60,000 psi and f'_c = 2500 psi

Required steel area (A_s) for positive moment M_A:

		Trial- 01		Trial- 02		Final	Controlling
Slab No.	**$A_{s\,min}$ (in^2/ft)**	**A_s (in^2/ft)**	**a (in)**	**A_s (in^2/ft)**	**a (in)**	**A_s (in^2/ft)**	**A_s (in^2/ft)**
01	0.0648	0.127004	0.298834	0.102944	0.242221	0.101393	0.101393
02	0.0756	0.126968	0.298747	0.108029	0.254185	0.107014	0.107014
03	0.0864	0.131733	0.309961	0.115758	0.272371	0.114998	0.114998
04	0.0864	0.159397	0.375053	0.141688	0.333383	0.140646	0.140646
05	0.0972	0.16568	0.389835	0.150387	0.353851	0.149572	0.149572
06	0.108	0.174311	0.410145	0.160765	0.37827	0.160092	0.160092
07	0.108	0.20216	0.475671	0.188073	0.442524	0.187248	0.187248
08	0.1188	0.211969	0.49875	0.199471	0.469343	0.198783	0.198783
09	0.1296	0.223384	0.525609	0.212199	0.499291	0.211611	0.211611
10	0.1296	0.252179	0.593363	0.241278	0.567713	0.240622	0.240622
11	0.1404	0.264708	0.622842	0.255087	0.600205	0.254532	0.254532
12	0.1512	0.278517	0.655334	0.270055	0.635424	0.269582	0.269582
13	0.1512	0.308606	0.726132	0.301109	0.708492	0.300639	0.300639

Required steel area (A_s) for negative moment M_A:

		Trial- 01		Trial- 02		Final	Controlling
Slab No.	**$A_{s\,min}$ (in^2/ft)**	**A_s (in^2/ft)**	**a (in)**	**A_s (in^2/ft)**	**a (in)**	**A_s (in^2/ft)**	**A_s (in^2/ft)**
01	0.0648	0.129678	0.305125	0.10529	0.247742	0.10368	0.10368
02	0.0756	0.129641	0.305036	0.110451	0.259884	0.109399	0.109399
03	0.0864	0.134507	0.316486	0.118331	0.278425	0.117543	0.117543
04	0.0864	0.162753	0.382948	0.144874	0.34088	0.143797	0.143797
05	0.0972	0.169168	0.398042	0.153743	0.361749	0.152903	0.152903

06	0.108	0.177981	0.418779	0.164336	0.386673	0.163643	0.163643
07	0.108	0.206416	0.485685	0.192288	0.452442	0.191441	0.191441
08	0.1188	0.216431	0.50925	0.203922	0.479816	0.203217	0.203217
09	0.1296	0.228087	0.536674	0.216919	0.510399	0.216319	0.216319
10	0.1296	0.257488	0.605855	0.246685	0.580435	0.246019	0.246019
11	0.1404	0.270281	0.635955	0.260787	0.613616	0.260226	0.260226
12	0.1512	0.28438	0.669131	0.276076	0.649591	0.275601	0.275601
13	0.1512	0.315103	0.741419	0.307866	0.72439	0.307401	0.307401

Required steel area (A_s) for positive moment M_B:

		Trial- 01		Trial- 02		Final	Controlling
Slab No.	**$A_{s\,min}$ (in^2/ft)**	**A_s (in^2/ft)**	**a (in)**	**A_s (in^2/ft)**	**a (in)**	**A_s (in^2/ft)**	**A_s (in^2/ft)**
01	0.0648	0.0752	0.176941	0.05901	0.138848	0.058428	0.0648
02	0.0756	0.071874	0.169115	0.059512	0.140028	0.059156	0.0756
03	0.0864	0.071885	0.169141	0.061642	0.145039	0.061388	0.0864
04	0.0864	0.084365	0.198505	0.07271	0.171081	0.072367	0.0864
05	0.0972	0.085456	0.201073	0.075414	0.177445	0.075153	0.0972
06	0.108	0.087943	0.206924	0.078993	0.185866	0.07878	0.108
07	0.108	0.100059	0.235434	0.090207	0.212251	0.089938	0.108
08	0.1188	0.103173	0.24276	0.094252	0.221769	0.094026	0.1188
09	0.1296	0.107136	0.252085	0.098916	0.232743	0.09872	0.1296
10	0.1296	0.119371	0.280872	0.110538	0.26009	0.110302	0.1296
11	0.1404	0.12384	0.291388	0.115645	0.272106	0.115437	0.1404
12	0.1512	0.128932	0.30337	0.121253	0.285302	0.121066	0.1512
13	0.1512	0.141504	0.332951	0.133414	0.313915	0.133196	0.1512

Required steel area (A_s) for negative moment M_B:

		Trial- 01		Trial- 02		Final	Controlling
Slab No.	**$A_{s\,min}$ (in^2/ft)**	**A_s (in^2/ft)**	**a (in)**	**A_s (in^2/ft)**	**a (in)**	**A_s (in^2/ft)**	**A_s (in^2/ft)**
01	0.0648	0.158756	0.373542	0.131331	0.309014	0.129035	0.129035
02	0.0756	0.151734	0.357021	0.130721	0.307579	0.129344	0.129344
03	0.0864	0.151757	0.357075	0.134466	0.316392	0.133504	0.133504
04	0.0864	0.178103	0.419067	0.159564	0.375445	0.158327	0.158327
05	0.0972	0.180407	0.424487	0.164617	0.387335	0.163692	0.163692
06	0.108	0.185657	0.43684	0.171833	0.404313	0.171097	0.171097
07	0.108	0.211237	0.497027	0.197076	0.463708	0.196205	0.196205
08	0.1188	0.21781	0.512493	0.205299	0.483057	0.20459	0.20459
09	0.1296	0.226176	0.532179	0.215	0.505883	0.214405	0.214405
10	0.1296	0.252005	0.592952	0.2411	0.567295	0.240445	0.240445
11	0.1404	0.26144	0.615153	0.251751	0.592356	0.2512	0.2512
12	0.1512	0.272191	0.640448	0.263575	0.620177	0.263106	0.263106
13	0.1512	0.298731	0.702896	0.290874	0.68441	0.290399	0.290399

Step 06: After that, we developed comparison graphs of: +A_s (in^2/ft) vs Short span (in), -A_s (in^2/ft) vs Short span (in), +A_s (in^2/ft) vs Long span (in), -A_s (in^2/ft) vs Long span (in), and a Combined graph for those 13 slab panel.

Finally we found the “Least Squares (R^2)” value and equation for each trend line of each graphs of calculated 13 slab panel.

Those graphs are given in the next chapter (Chapter 04: Results and Discussion).

3.2 Beam

3.2.1 Design steps of simplified beam design (Flexure design)

Step 01: Determine minimum thickness of beam.

Determine minimum thickness, t_{min} of beam according to ACI Code 318-11, section 9.5.2.1 (Table 2.7).

Step 02: Calculate $d_{given.}$ $d_{given} = t_{min}$ (roundup) – 2.5 in. (For singly reinforced beam)

Step 03: Calculation of factored load.

W_{DL}= 1.2*DL and W_{LL}= 1.6*LL;

$W_u = W_{DL}+W_{LL}$

Where DL= Total dead load (i.e.: Slab self weight on beam, Self weight of beam, Floor finish, Partition wall, Plaster etc.)

LL= Live load.

Step 04: To design a beam maximum bending moments at mid span and supports are required. Moments can be calculated from exact analysis (i.e. moment distribution, slope deflection method etc) using finite element software package. But approximate bending moments can be calculated from ACI 318-11, section 8.3.3. Moment coefficients are given in the Table 2.2.

Step 05: Calculation of steel ratio, ρ

$$\rho_{max} = 0.85\beta_1 \frac{f_c'}{f_y} \frac{\varepsilon_u}{\varepsilon_u + 0.004} \quad \text{Here, } \varepsilon_u = 0.003.$$

Step 06: Calculation of effective flange width, b.

As slab and beams are casted monolithically it is permitted to include the contribution of the slab in beam. Effective width of the flange can be calculated as per ACI 318-11, section 8.10.2 which is given in the Table 2.6.

Step 07: Determination of $d_{req.}$

$R_n = \rho\, f_y (1- \frac{0.5\rho f_y}{0.85 f'_c})$

Again, $(bd^2)_{req} = \frac{M_u}{\emptyset R_n}$ So, $d_{req} = \sqrt{\frac{M_u}{\emptyset b R_n}}$

If, $d_{provided} > d_{req}$ (Ok). Otherwise, increase beam depth.

Step 08: Determine required steel area.

$A_s = \frac{M_u}{\emptyset * f_y * (d - \frac{a}{2})}$; Here, Ø= 0.9

And $a = \frac{A_s * f_y}{0.85 * f'_c * b}$

According to ACI 318-11, section 10.5 minimum tensile reinforcement should be provided to resist the cracking moment. For any section minimum reinforcement can be calculated by the equation:

$$(A_s)_{min} = \frac{3\sqrt{f_c'}}{f_y} b_w d \geq \frac{200}{f_y} b_w d,$$ Where f_c' and f_y are in psi.

3.2.2 Comparison table of required steel area for full roh and half roh of beam

At first we selected 7 beams randomly. Then we calculated required steel area (A_s) for both positive and negative moments and for $\rho = \rho^{max}$ and $\rho = 0.5\times\rho^{max}$ for different load conditions, following step 01 to step 08. Then we make a summary in a table for comparison.

Here, L= Length of span of beam (ft or m)

b_w= Bottom width of beam (in or mm); S.T.= Slab thickness (in or mm)

d= Effective depth of beam (in or mm); a= Stress block depth (in or mm).

Table 3.4: Comparison table for required steel area for full roh and half roh of beam.

Beam Details	**d (in) or (mm)**		**$A_{s\,min}$ (in^2) or (mm^2)**	**+ A_s (in^2) or (mm^2)**		**- A_s (in^2) or (mm^2)**		**a (in) or (mm)**
	$\rho=\rho_{max}$	**$\rho=0.5\rho_{max}$**		**$\rho=\rho_{max}$**	**$\rho=0.5\rho_{max}$**	**$\rho=\rho_{max}$**	**$\rho=0.5\rho_{max}$**	
L =19 ft; b_w=12 in; S.T.=6 in.	10	10	0.39	2.35	3.73	2.35	3.73	0.011099999999999714
L =19 ft; b_w=12 in; S.T.=6 in.	10	10	0.39	1.88	2.96	1.88	2.96	0.008889999999999804
L =16 ft; b_w=12 in; S.T.=6 in.	8	8	0.32	2.07	3.29	2.07	3.29	0.009699999999999771
L =16 ft; b_w=15 in; S.T.=6 in.	8	8	0.39	2.11	3.36	2.11	3.36	0.009899999999999763
L =20 ft; b_w=15 in; S.T.=8 in.	10.5	10.5	0.52	2.79	4.44	2.79	4.44	0.01316999999999963

Beam Details	d (in) or (mm)		$A_{s\ min}$ (in^2) or (mm^2)	+ A_s (in^2) or (mm^2)		- A_s (in^2) or (mm^2)		a (in) or (mm)
	$\rho = \rho_{max}$	$\rho = 0.5\rho_{max}$		$\rho = \rho_{max}$	$\rho = 0.5\rho_{max}$	$\rho = \rho_{max}$	$\rho = 0.5\rho_{max}$	
L = 5 m; b_w=300 mm; S.T.=150 mm.	206.5	206.5	206.70	1124.49	1779.57	1124.49	1779.57	0.008349999999999826
L = 5 m; b_w=300 mm; S.T.=150 mm.	176.5	176.5	174.53	1589.22	2549.11	1589.22	2549.11	0.011589999999999694

Chapter 04
RESULTS AND DISCUSSION

A JavaScript programming language based website has been created for simplified design of reinforced concrete buildings of moderate size and height. The user including the engineer, the architect, and common non-technical person can give very simple inputs, e.g. slab width, length etc. in this webpage and instantly get the visual results there. It can be used for initial structural design, verifying existing design and detail estimation of materials. Link of this website is given below:

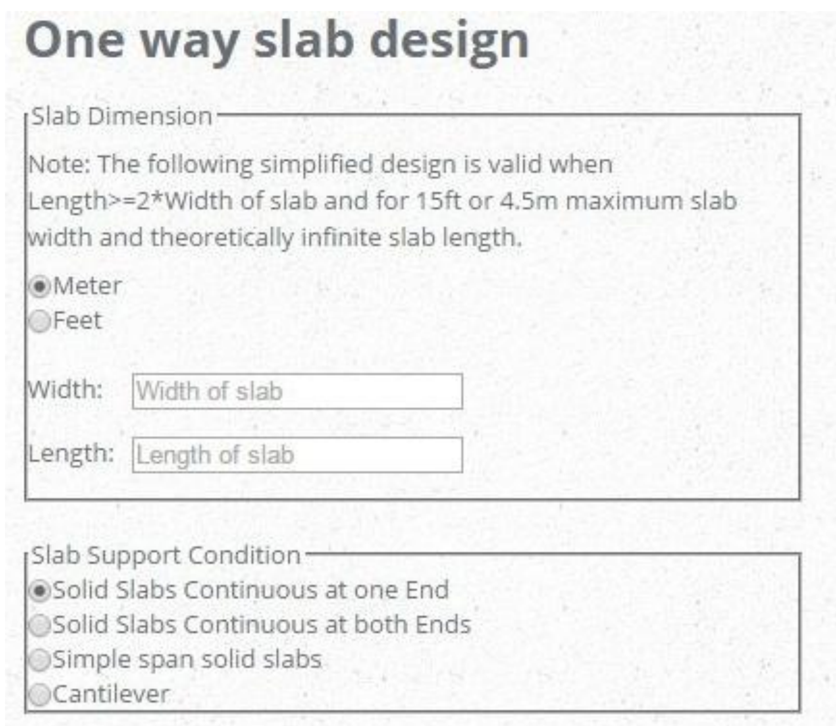
http://simplifieddesignofconcretestructures.weebly.com

4.1 One way slab

4.1.1 Simplified design of one way slab

User needs to input the following information about the slab i.e. slab dimensions, slab support condition, yield strength and diameter of the steel in order to analyze the slab.

This analysis includes thickness of the slab, area of required steel and spacing of the rebar. Finally plan is shown showing the reinforcement detailing of the slab.



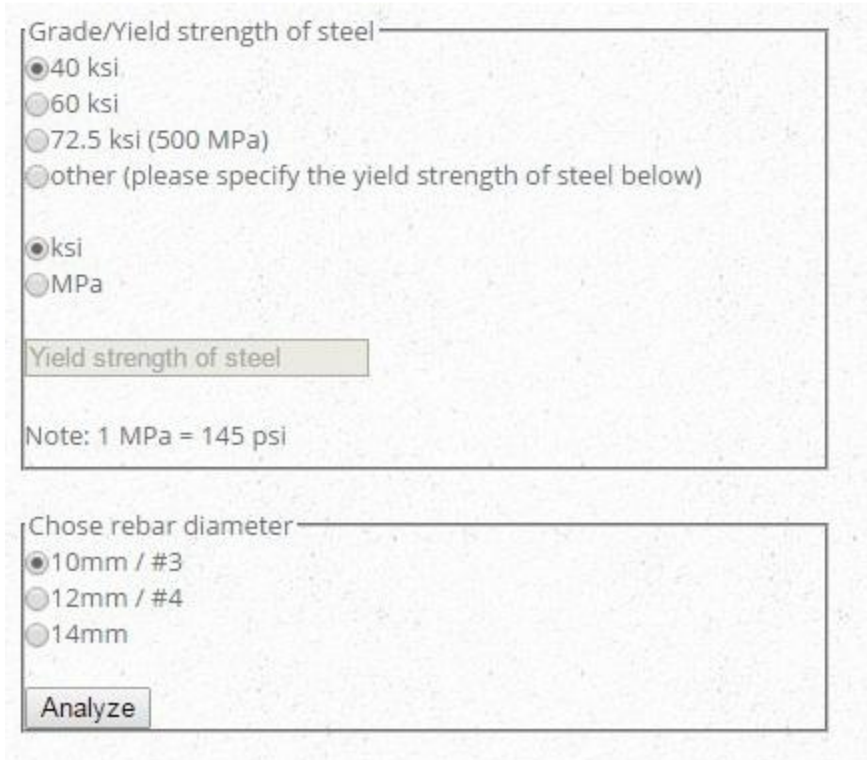


Figure 4.1: One way slab design webpage screenshot.

This webpage is uploaded to the following link:

http://simplifieddesignofconcretestructures.weebly.com/one-way-slab-design.html

4.2 Two way slab

4.2.1 Least Squares (R^2) values of graphs for slab thickness Determination

From alpha calculation (Article 3.1.1), we can see that, for all floor slabs of the plan values of alpha, $\alpha_{avg} > 2$. So according to Article 2.2.1 we use ACI Eq. 9-13 for determining minimum slab thickness. That is given below:

For α_m greater than 2.0, the thickness must not be less than

$$h = \frac{l_n\left(0.8+\left(\frac{f_y}{200,000}\right)\right)}{36+9\beta}$$ and not less than 3.5 inch….. (2.2) (ACI Eq. 9-13)

Where l_n = clear span in long direction, in.

α_m = average value of α for all beams on edges of a panel. [$\alpha = \frac{\mathbf{E}_{cb}\,\mathbf{I}_b}{\mathbf{E}_{cs}\,\mathbf{I}_s}$]

β = ratio of clear span in long direction to clear span in short direction.

and f_y = 60,000 psi.

Now, R^2 (Least Squares) values of different slab thicknesses determination graphs are given below in a table:

Table 4.1: R^2 (Least Squares) values of graphs for slab thickness determination.

Figure No.	Name of the diagram	R^2
3.1	$\frac{P_{Inner}}{145}$ (in) vs Thickness corresponding α, (in) diagram	0.970
3.2	$\frac{Pc/c}{145}$ (in) vs Thickness corresponding α, (in) diagram	0.97
3.3	$\frac{P_{Outer}}{145}$ (in) vs Thickness corresponding α, (in) diagram	0.970
3.4	$\frac{P}{150}$ (in) vs Thickness corresponding α, (in) diagram	0.970
3.5	$\frac{P}{160}$ (in) vs Thickness corresponding α, (in) diagram	0.969
3.6	$\frac{P}{180}$ (in) vs Thickness corresponding α, (in) diagram	0.97

Though all the R^2 values are 0.970 or near to 0.970, we will use $\frac{P_{Outer}}{145}$ for thickness determination as we don't know the beam dimensions for preliminary design and the design will be safe design.

$$\textbf{Thickness (in)} = \frac{\textbf{Perimeter}}{\textbf{145}}$$

4.2.2 Least Squares (R^2) values of graphs for the relation between length and steel area

Length vs steel area relationship graphs are given below:

R^2 (Least Squares) values of length vs steel area relationship graphs are incorporated in a table:

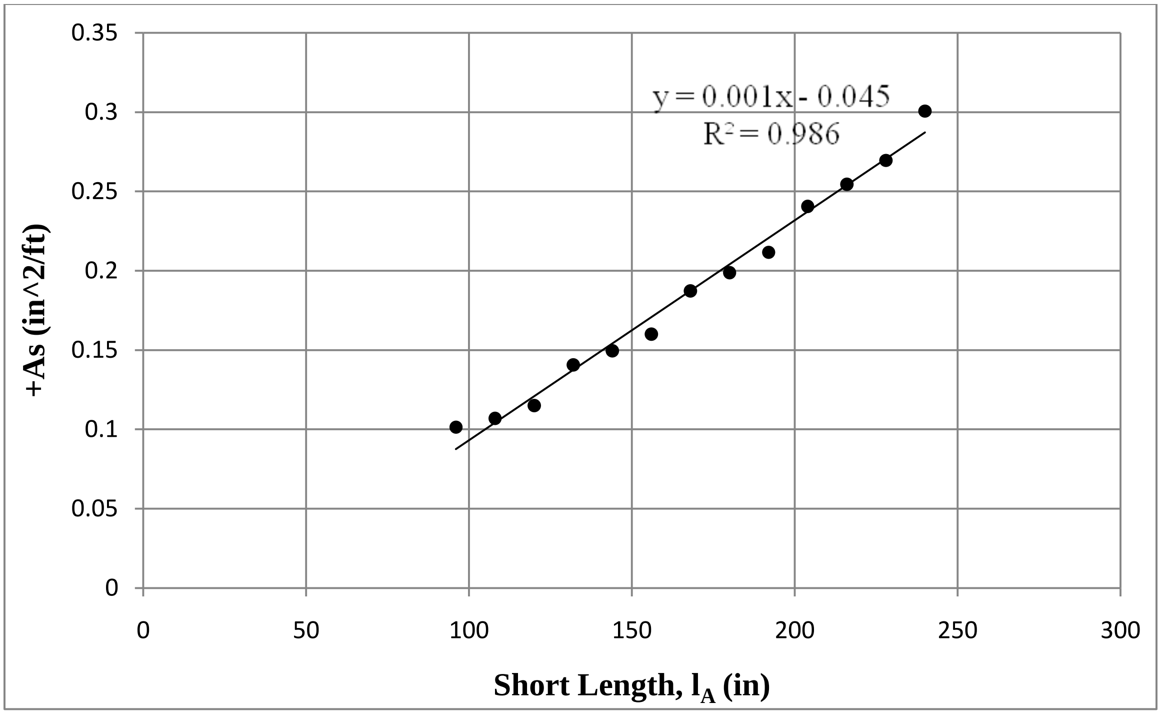


Figure 4.2: $+A_s$ (in^2/ft) vs Short span (in) diagram. (Linear trend line)

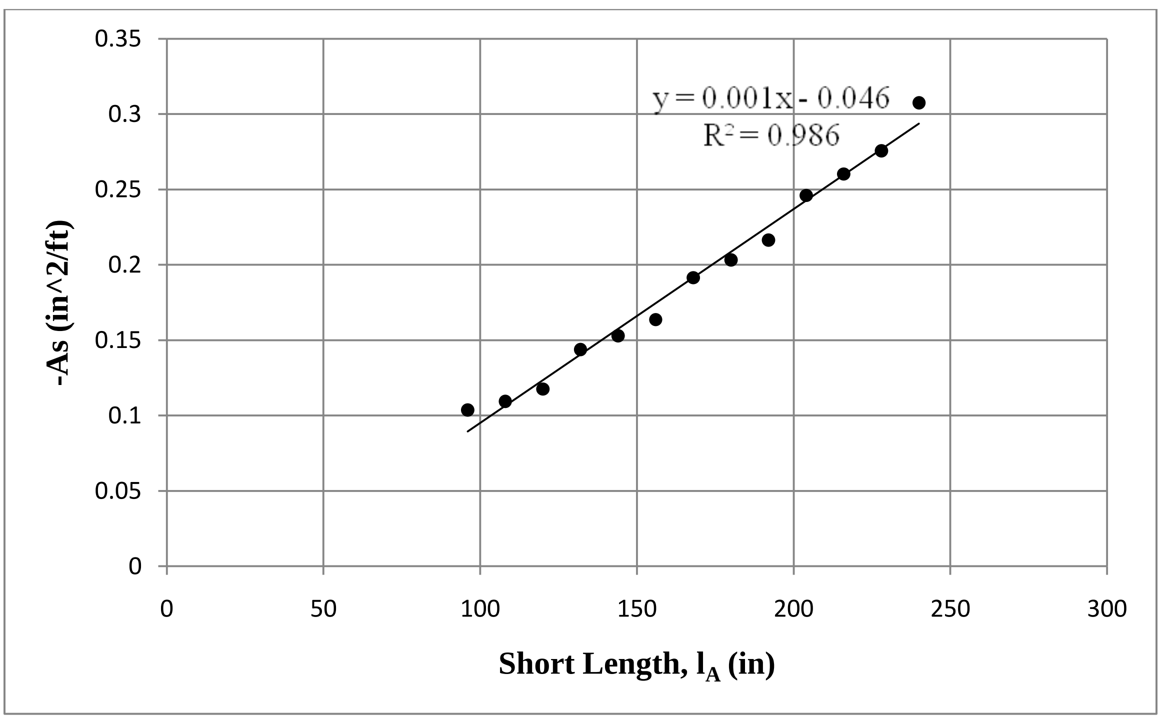


Figure 4.3: $-A_s$ (in^2/ft) vs Short span (in) diagram. (Linear trend line)

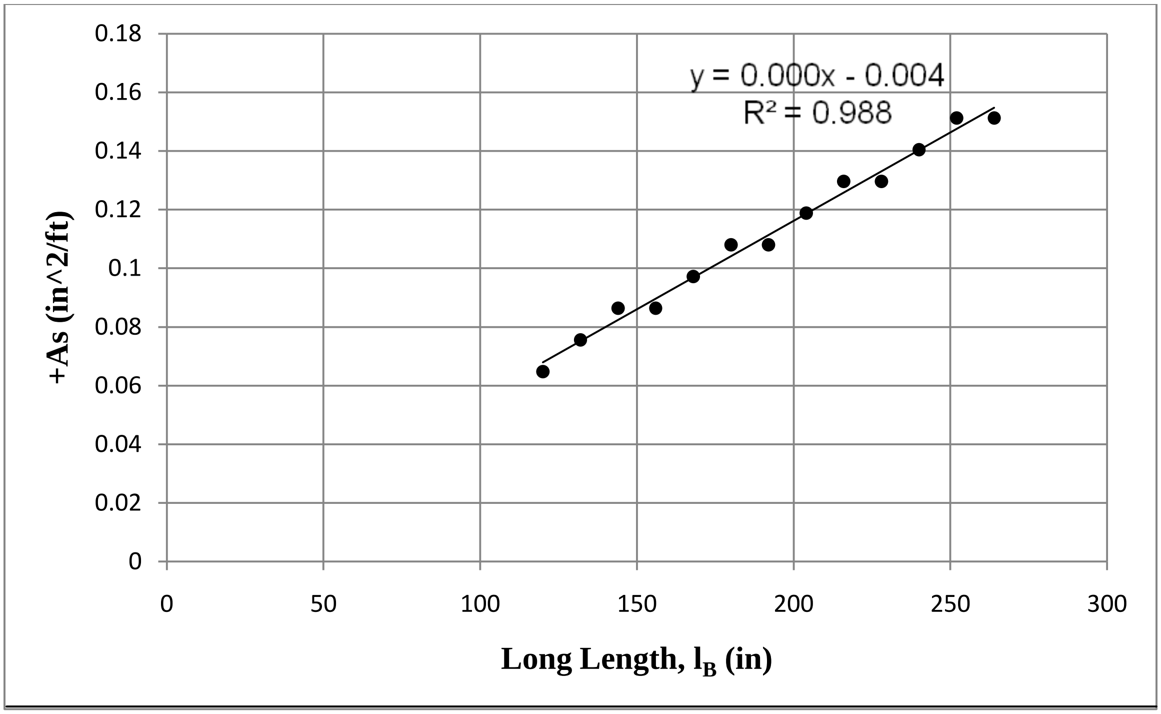


Figure 4.4: +A_s (in^2/ft) vs Long span (in) diagram. (Linear trend line)

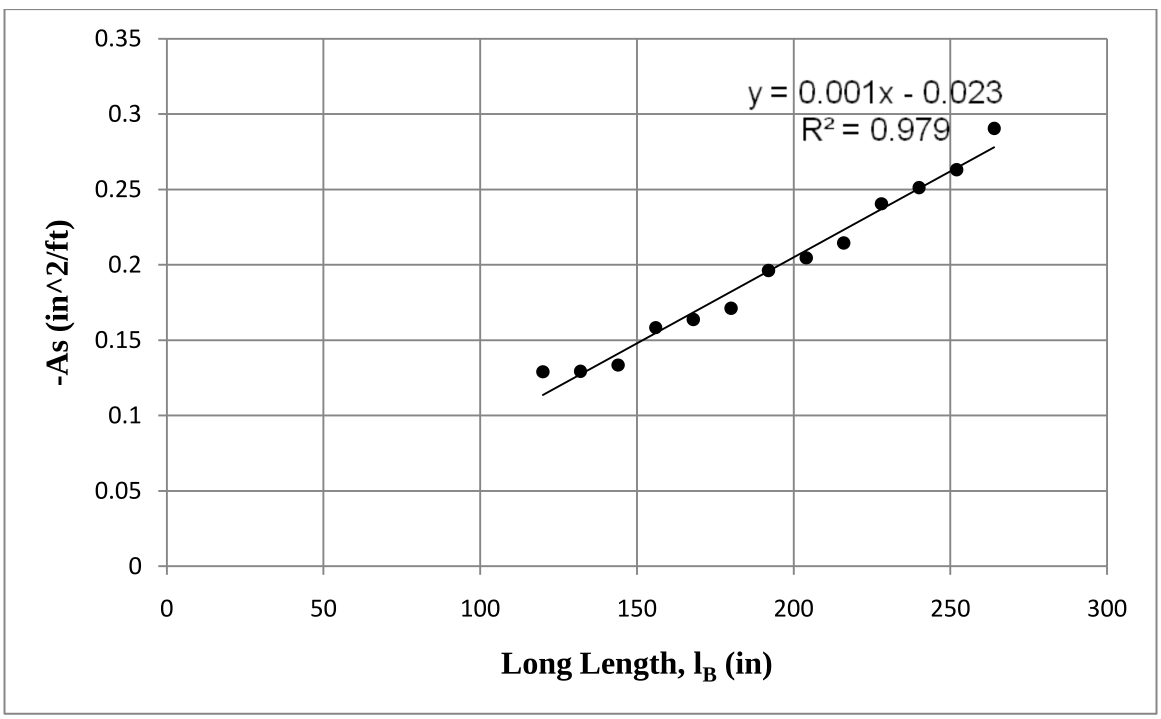


Figure 4.5: -A_s (in^2/ft) vs Long span (in) diagram. (Linear trend line)

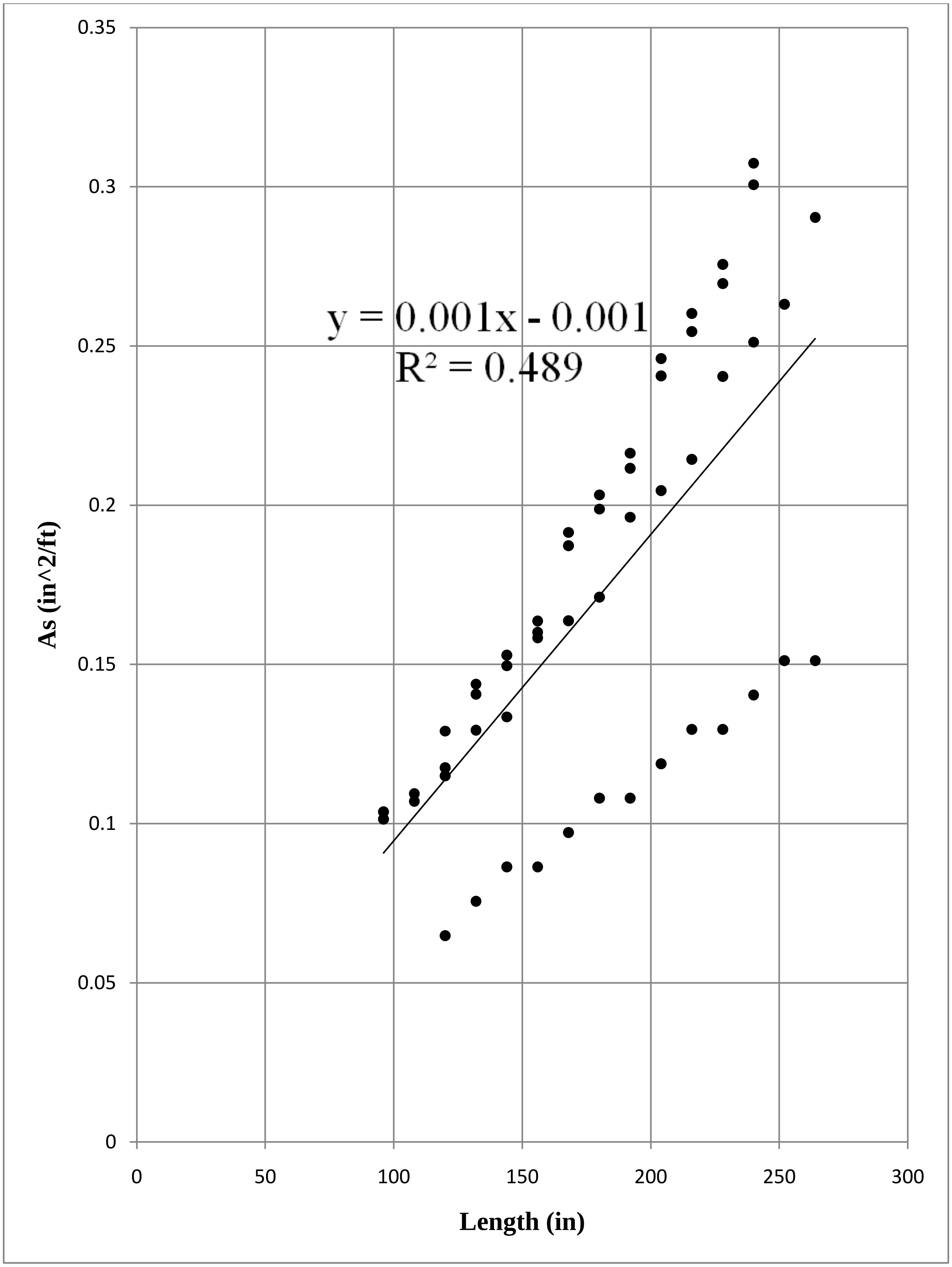


Figure 4.6: Combined diagram of A_s (in^2/ft) vs Span length (in). (Linear trend line)

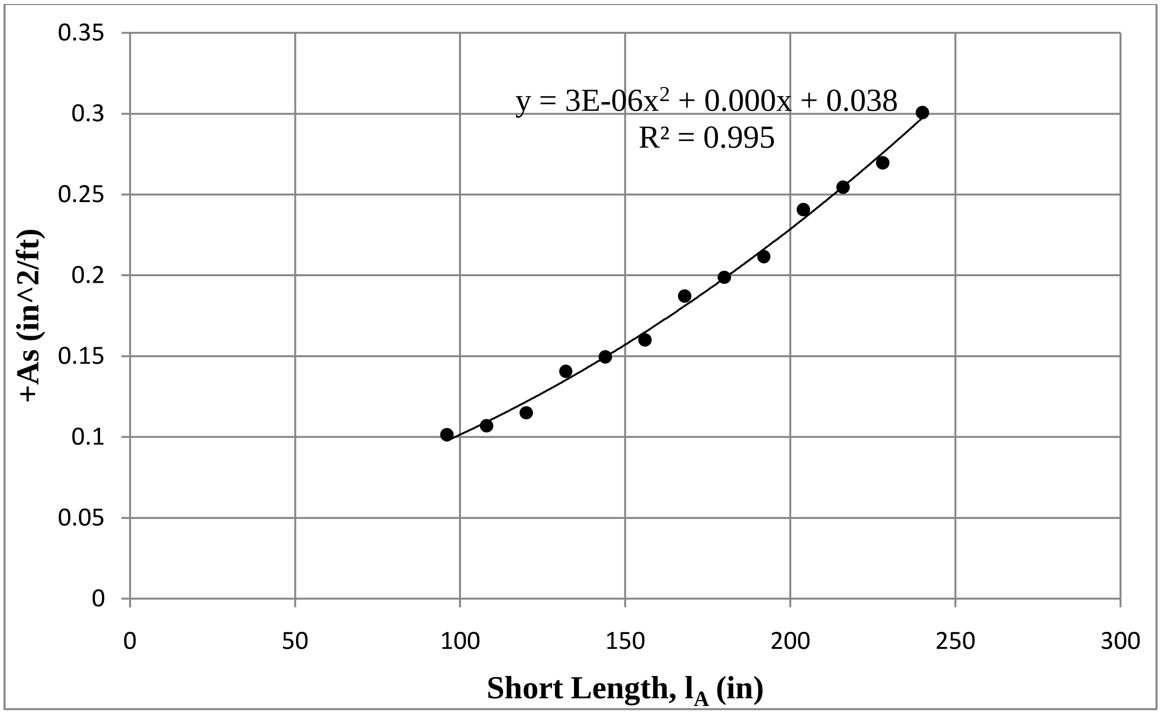


Figure 4.7: +A_s (in^2/ft) vs Short span (in) diagram. (Polynomial trend line, order 2)

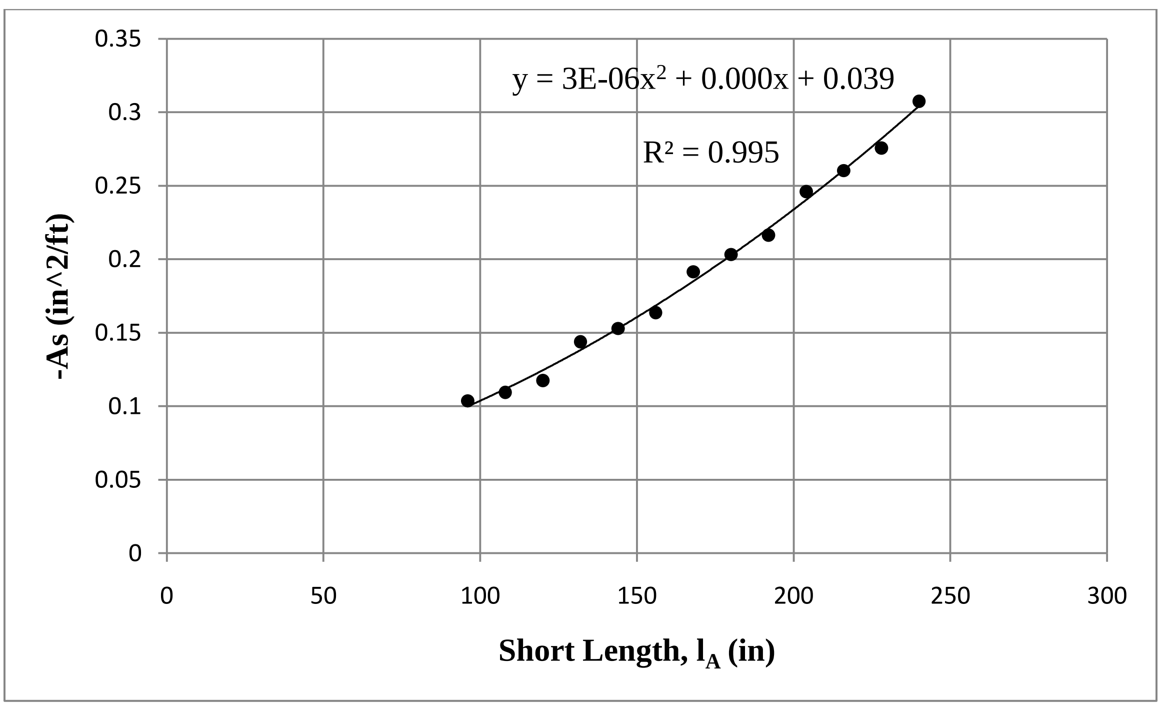


Figure 4.8: -A_s (in^2/ft) vs Short span (in) diagram. (Polynomial trend line, order 2)

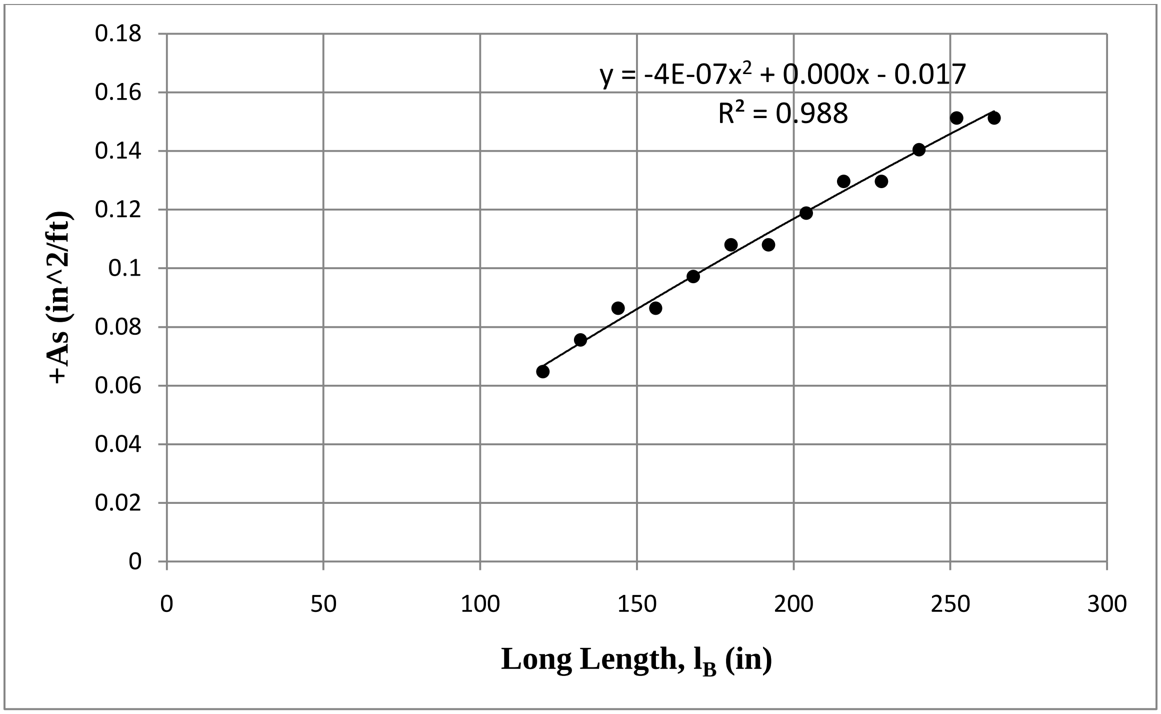


Figure 4.9: $+A_s$ (in^2/ft) vs Long span (in) diagram. (Polynomial trend line, order 2)

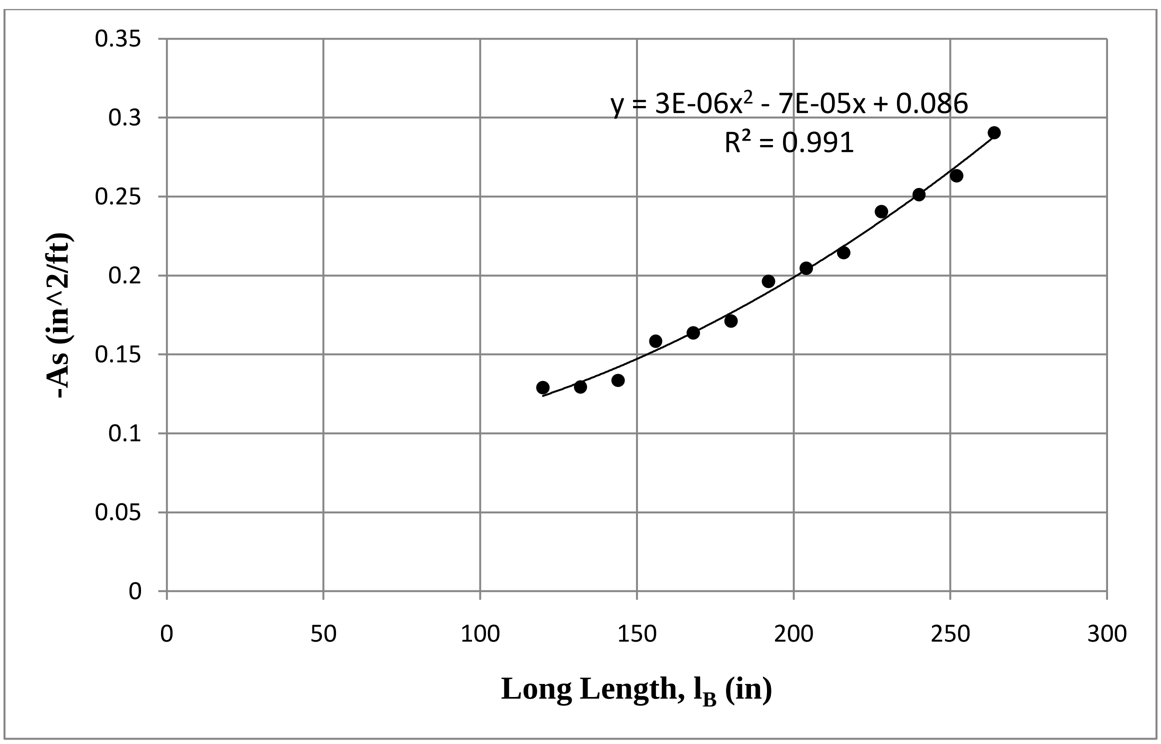


Figure 4.10: $-A_s$ (in^2/ft) vs Long span (in) diagram. (Polynomial trend line, order 2)

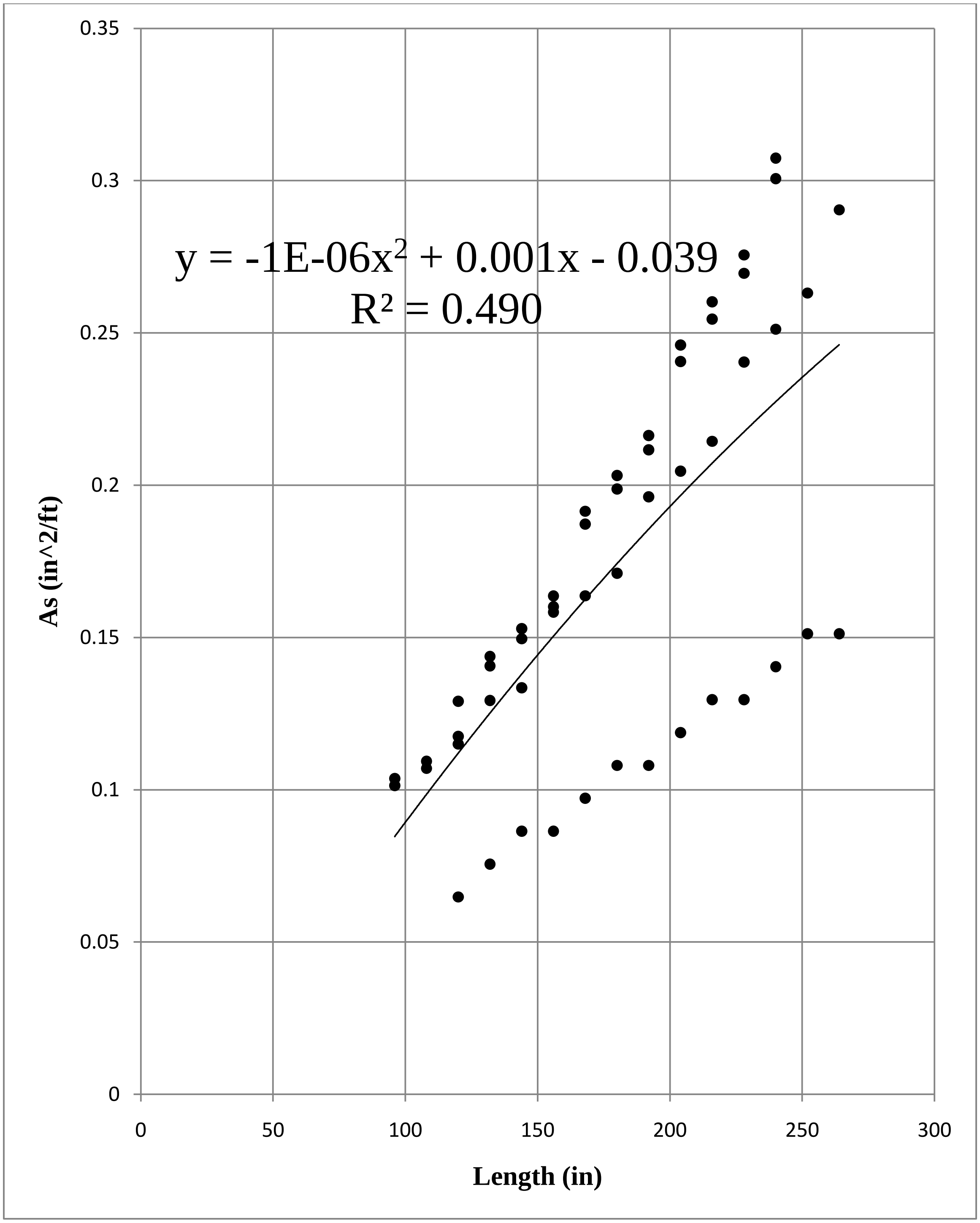


Figure 4.11: Combined diagram of A_s (in^2/ft) vs Span length (in). (Polynomial trend line, order 2)

Table 4.2: R^2 (Least Squares) values of graphs for the length vs steel area relationship (Linear trend line).

Figure No	**Name of the diagram**	**Equation of the trend line**	$\mathbf{R^2}$
4.2	$+A_s$ (in^2/ft) vs Short span (in) diagram	$y = 0.001x - 0.045$	0.986
4.3	$-A_s$ (in^2/ft) vs Short span (in) diagram	$y = 0.001x - 0.046$	0.986
4.4	$+A_s$ (in^2/ft) vs Long span (in) diagram	$y = 0.000x - 0.004$	0.988
4.5	$-A_s$ (in^2/ft) vs Long span (in) diagram	$y = 0.001x - 0.023$	0.979
4.6	Combined diagram of A_s (in^2/ft) vs Span length (in)	$y = 0.001x - 0.001$	0.489

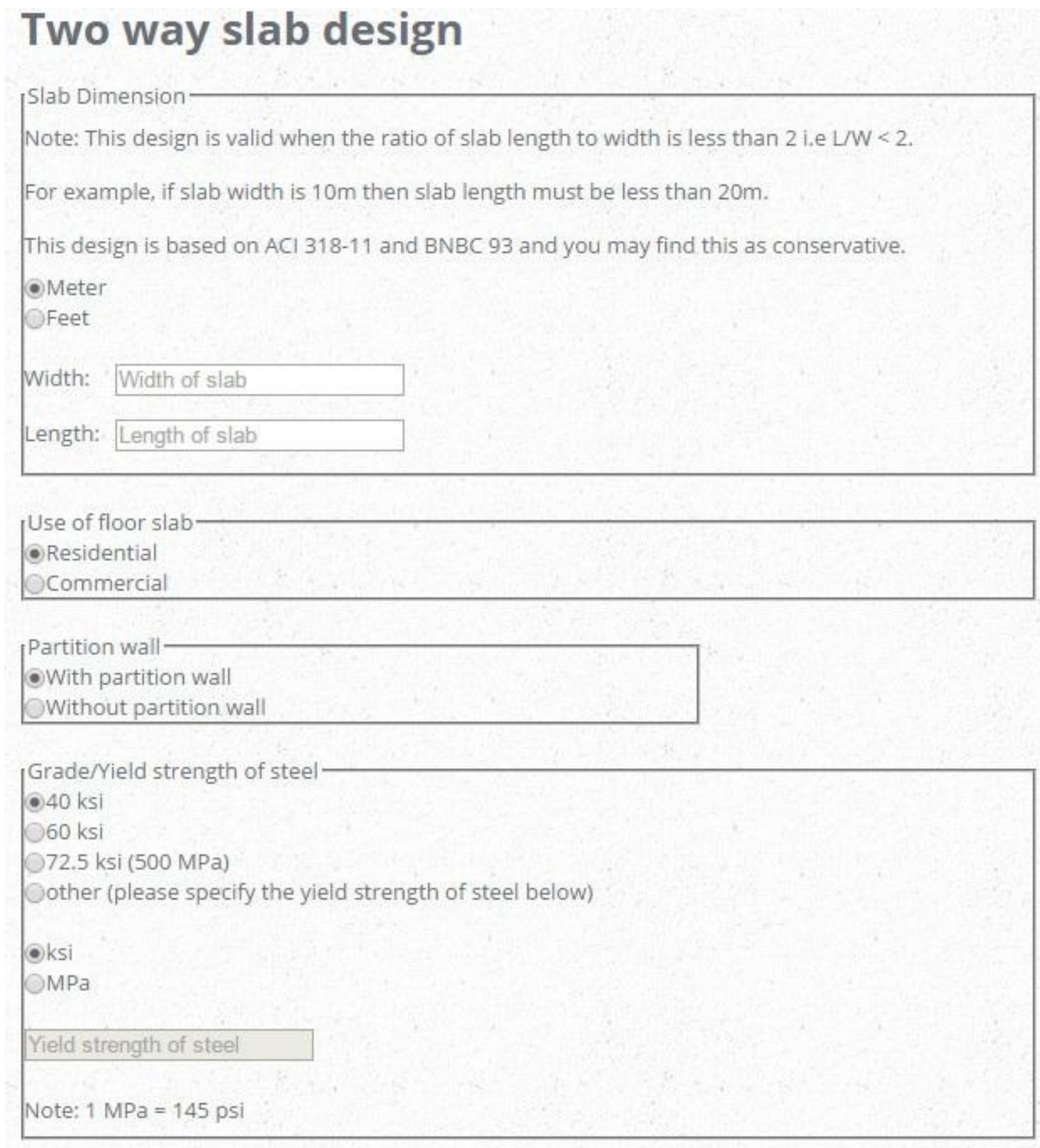
Table 4.3: R^2 (Least Squares) values of graphs for the length vs steel area relationship (Polynomial trend line, order 2).

Figure No	**Name of the diagram**	**Equation of the trend line**	$\mathbf{R^2}$
4.7	$+A_s$ (in^2/ft) vs Short span (in) diagram	$y = 3E\text{-}06x^2 + 0.000x + 0.038$	0.995
4.8	$-A_s$ (in^2/ft) vs Short span (in) diagram	$y = 3E\text{-}06x^2 + 0.000x + 0.039$	0.995
4.9	$+A_s$ (in^2/ft) vs Long span (in) diagram	$y = -4E\text{-}07x^2 + 0.000x - 0.017$	0.988
4.10	$-A_s$ (in^2/ft) vs Long span (in) diagram	$y = 3E\text{-}06x^2 - 7E\text{-}05x + 0.086$	0.991
4.11	Combined diagram of A_s (in^2/ft) vs Span length (in)	$y = -1E\text{-}06x^2 + 0.001x - 0.039$	0.490

4.2.3 Simplified design of a two way slab

User needs to input the following information about the slab i.e. slab dimensions, occupancy of the slab, various loading conditions on slab, yield strength and diameter of the steel, properties of concrete and concrete mix ratio in order to analyze the slab.

This analysis includes thickness of the slab, area of required steel and spacing of the rebar. Finally plan is shown showing the reinforcement detailing of the slab. It also shows the quantity of constituents used in the mixture.



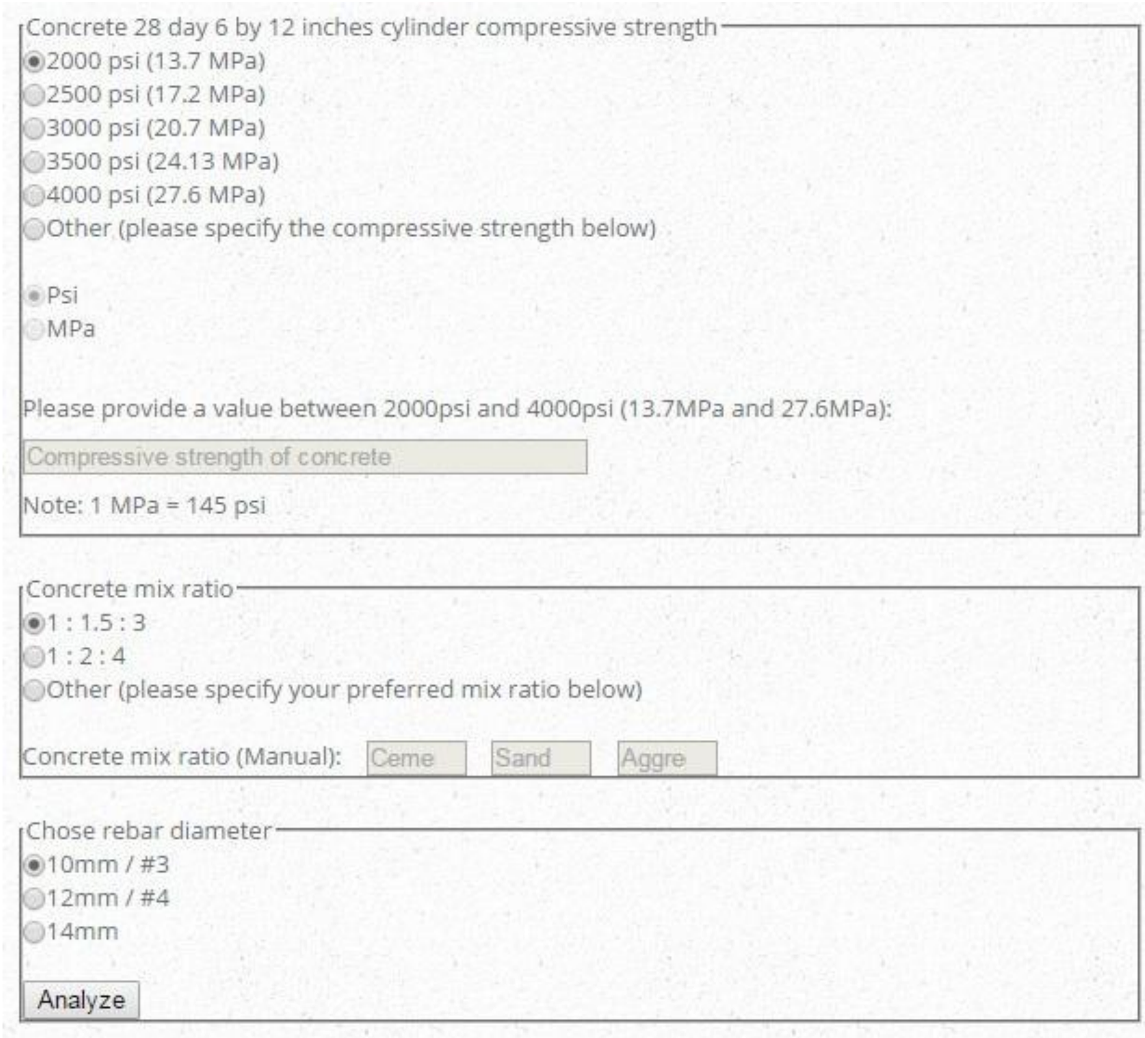


Figure 4.12: Two way slab design webpage screenshot.

This webpage is uploaded to the following link:

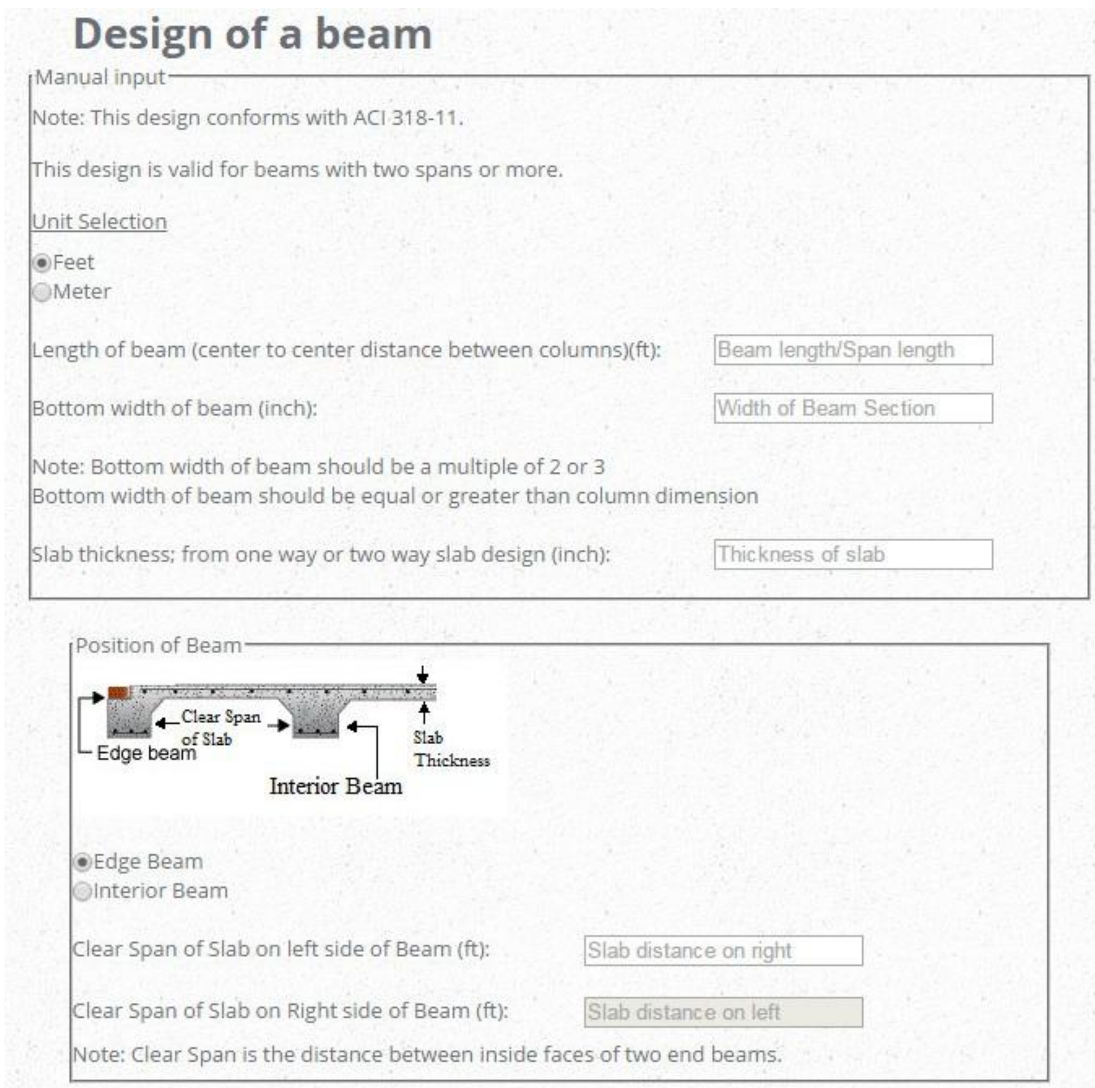
http://simplifieddesignofconcretestructures.weebly.com/two-way-slab-design.html

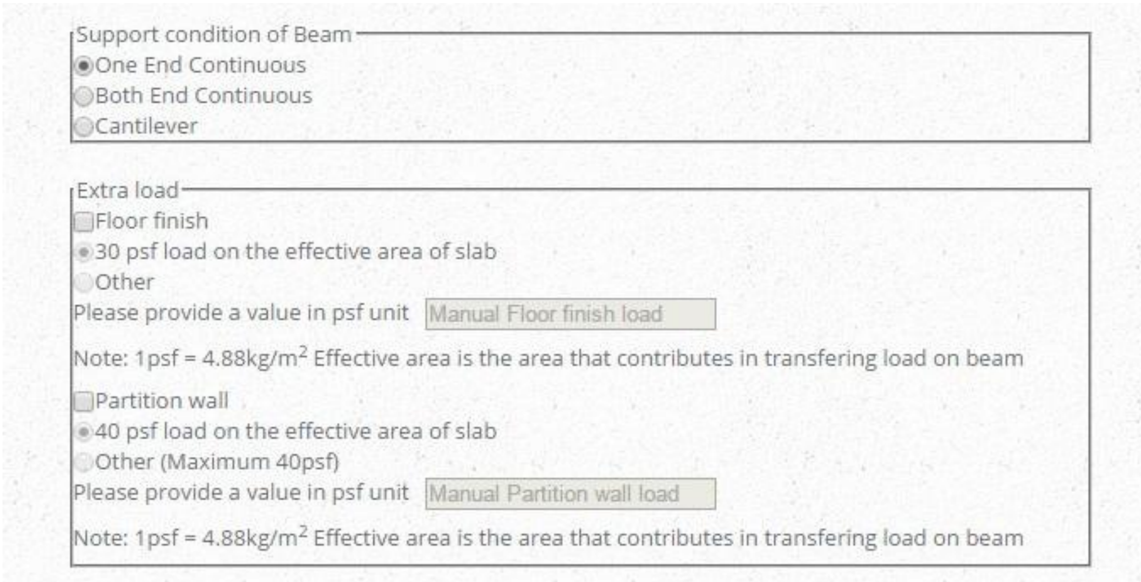
4.3 Beam

4.3.1 Simplified design of a beam (Flexure design)

User needs to input the following information about the beam i.e. beam dimensions, beam support condition, position of the beam, various loading condition on the beam, slab thickness, yield strength and diameter of the steel and the proportion of various constituents used in the mixture in order to analyze the beam.

This analysis shows area of steel required.




Support condition of Beam
One End Continuous
Both End Continuous
Cantilever
Extra load
Floor finish
30 psf load on the effective area of slab
Other
Please provide a value in psf unit
Manual Floor finish load
Note: 1psf = 4.88kg/m2 Effective area is the area that contributes in transfering load on beam
Partition wall
40 psf load on the effective area of slab
Other (Maximum 40psf)
Please provide a value in psf unit
Manual Partition wall load
Note: 1psf = 4.88kg/m2 Effective area is the area that contributes in transfering load on beam

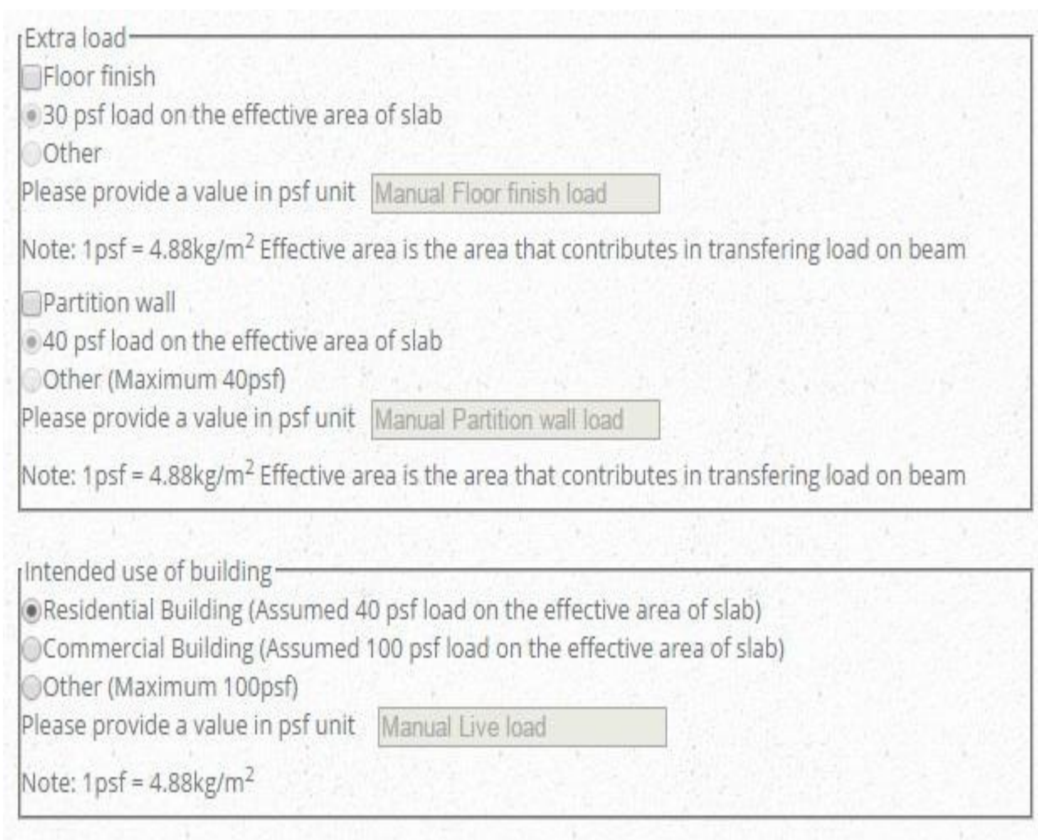

Extra load
Floor finish
30 psf load on the effective area of slab
Other
Please provide a value in psf unit
Manual Floor finish load
Note: 1psf = 4.88kg/m2 Effective area is the area that contributes in transfering load on beam
Partition wall
40 psf load on the effective area of slab
Other (Maximum 40psf)
Please provide a value in psf unit
Manual Partition wall load
Note: 1psf = 4.88kg/m2 Effective area is the area that contributes in transfering load on beam
Intended use of building
Residential Building (Assumed 40 psf load on the effective area of slab)
Commercial Building (Assumed 100 psf load on the effective area of slab)
Other (Maximum 100psf)
Please provide a value in psf unit
Manual Live load
Note: 1psf = 4.88kg/m2

Figure 4.13: Beam design webpage screenshot.

This webpage is uploaded to the following link:

http://simplifieddesignofconcretestructures.weebly.com/beam-design.html

4.3.2 Maximum and minimum no. of bars in a single layer of beam

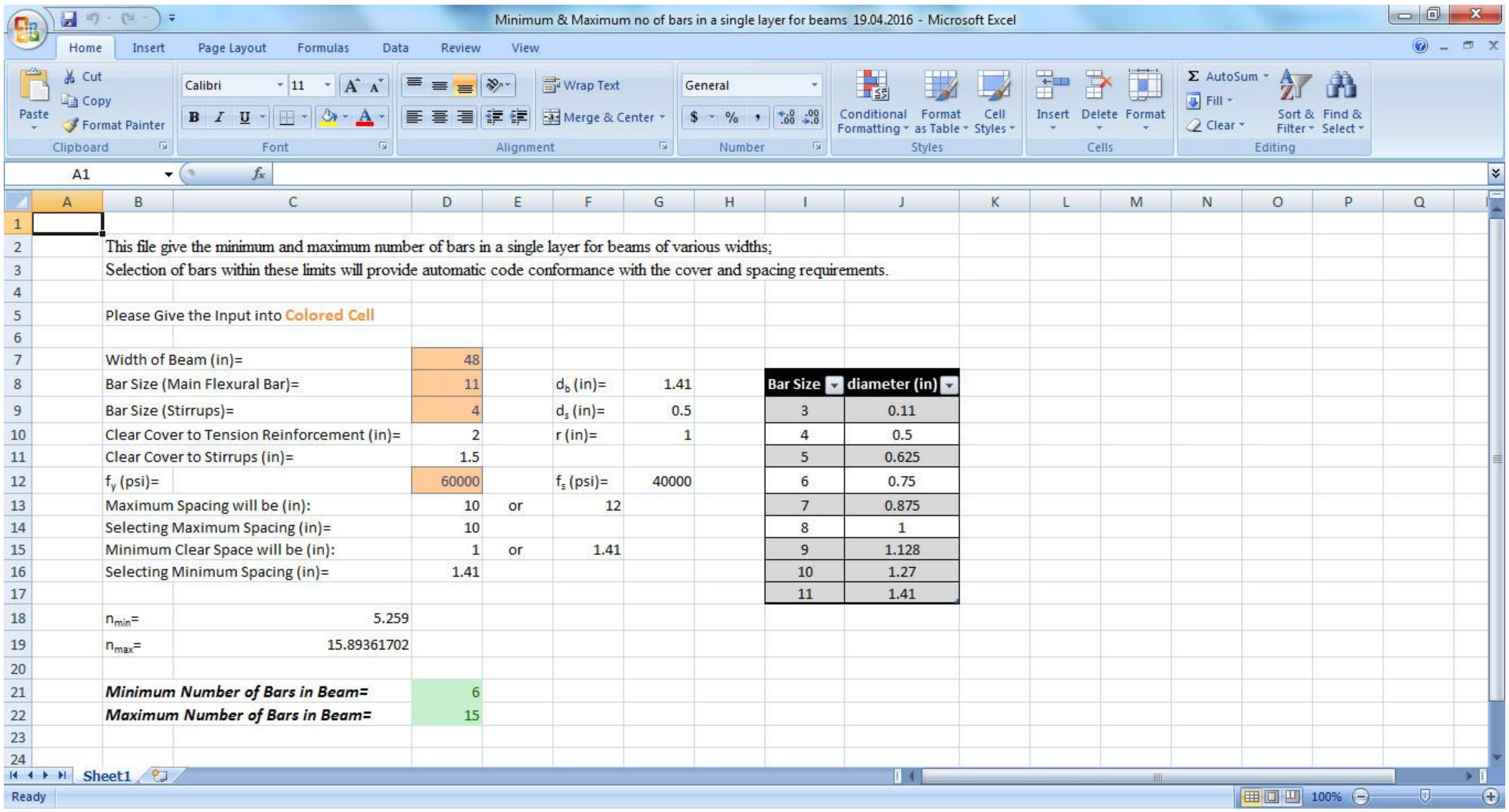


Figure 4.14: Maximum and minimum no. of bars in a single layer of beam excel file screenshot.

4.3.3 Simplified design of beam (Shear design)

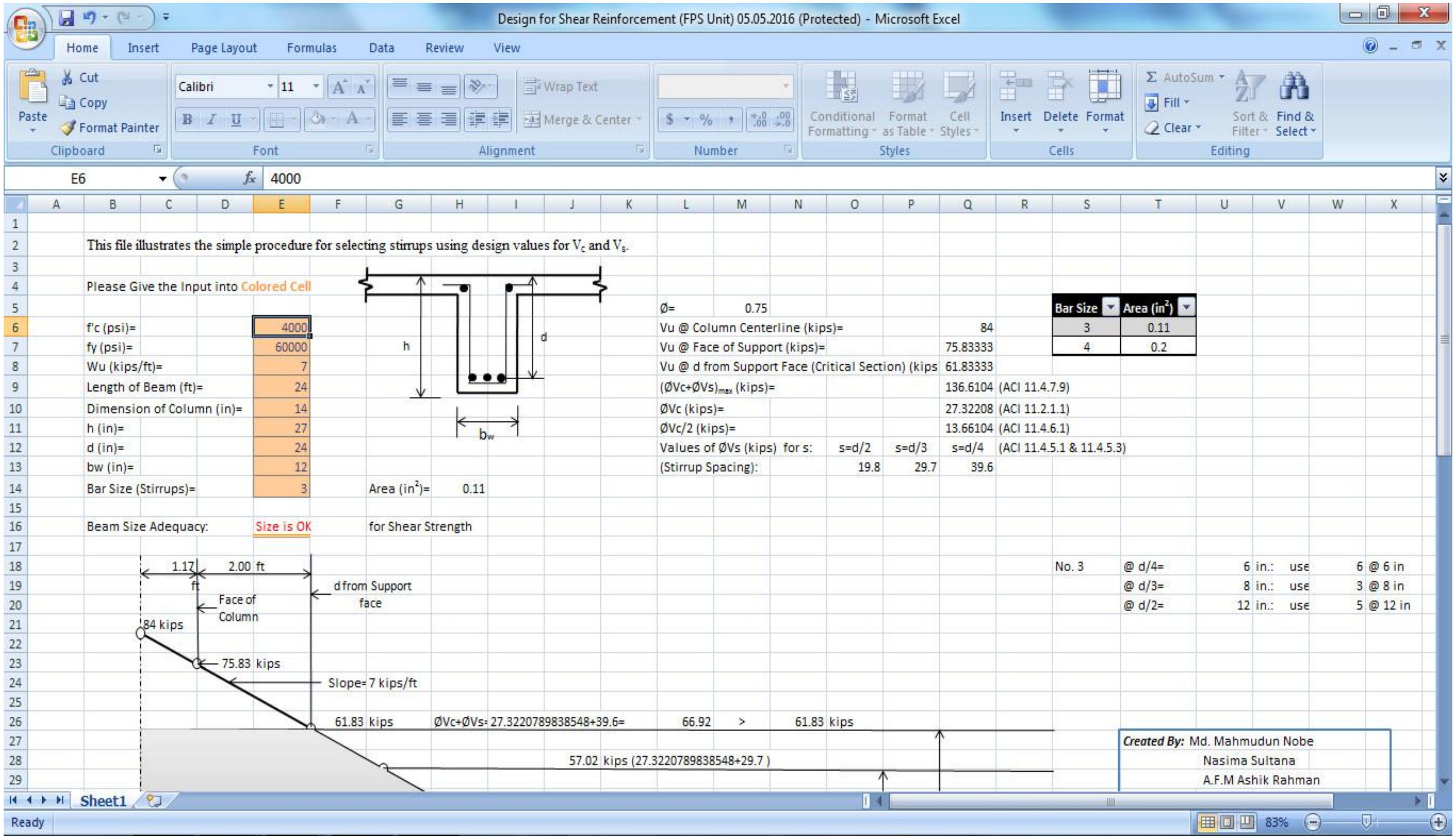


Figure 4.15 (a): Design of shear of beam excel file screenshot.

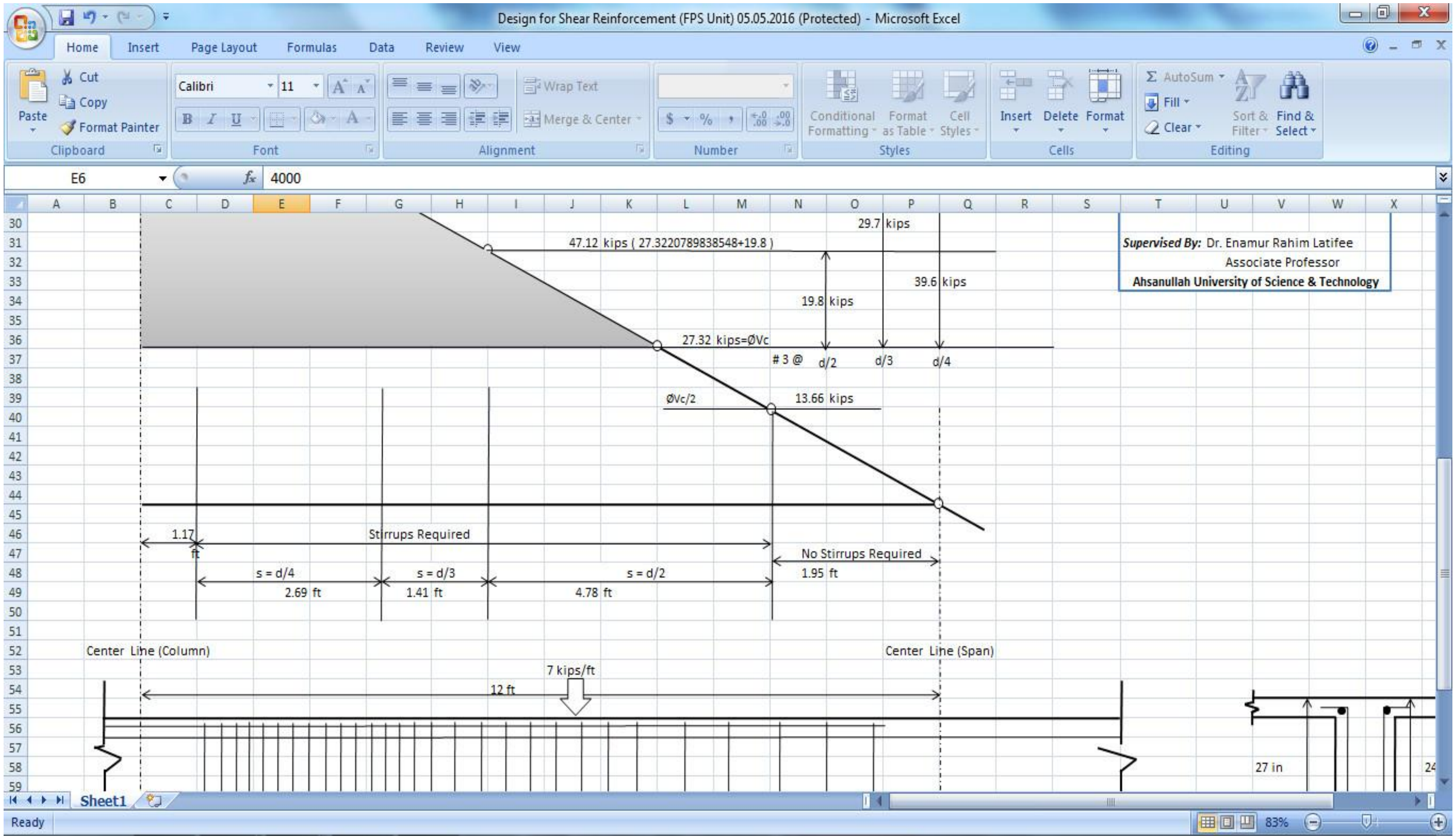


Figure 4.15 (b): Design of shear of beam excel file screenshot.

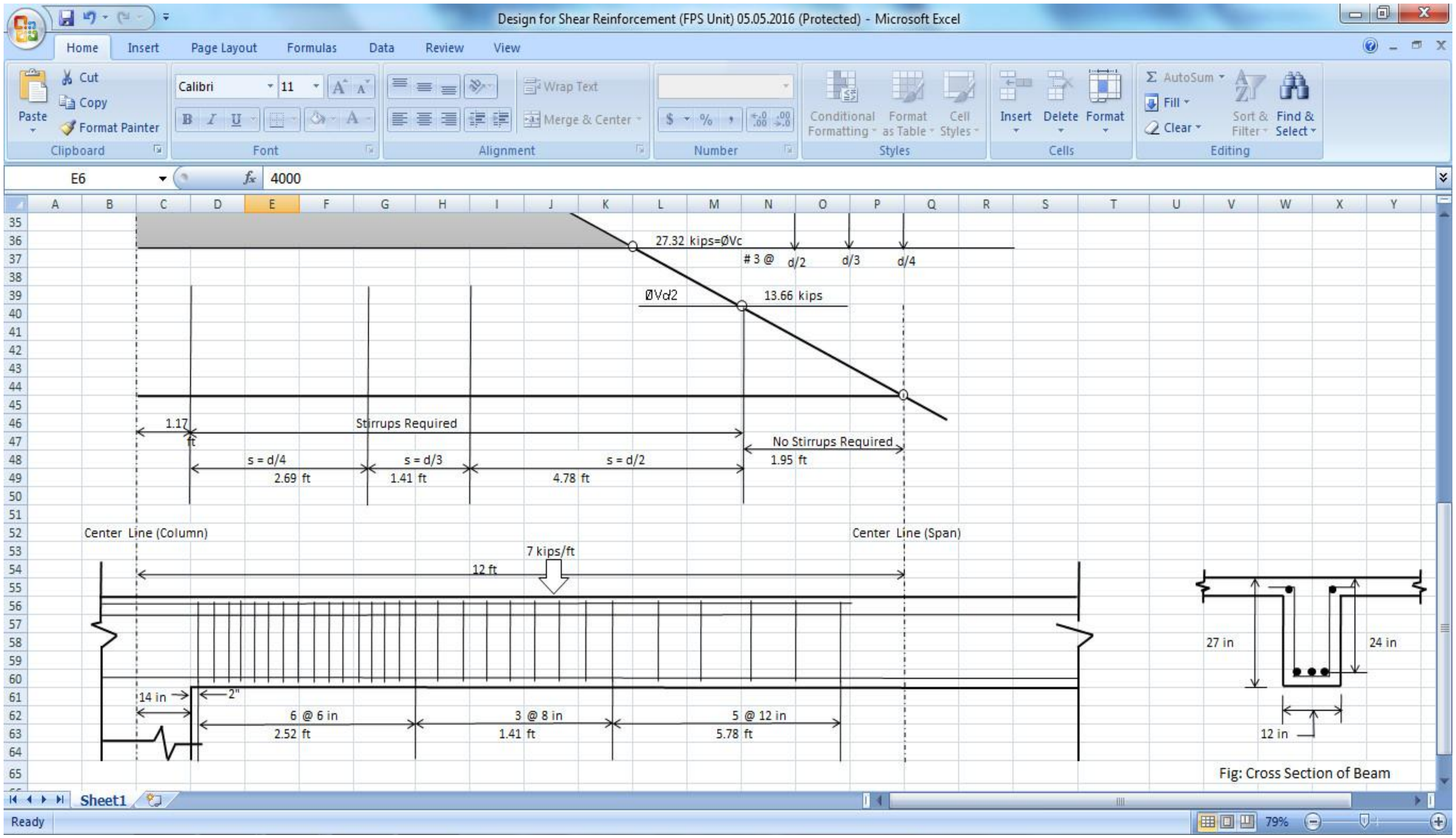


Figure 4.15 (c): Design of shear of beam excel file screenshot.

Chapter 05
CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

The purpose of this paper is to give practicing engineers some way of reducing the design time required for smaller projects, while still complying with the letter & intent of the ACI Standard 318, Building Code Requirements for Structural Concrete. Here design load & live load are considered in accordance of the code. If wind load, resistance to earthquake, induced forces, earth or liquid pressure, impact effects or structural effects of differential settlement need to be included in the design, such effects should be considered separately. They are not included within the scope of simplified design techniques presented here.

This simplified design approach can be used for conventionally reinforced concrete buildings of moderate size & height with usual spans & story height. This paper was prepared for the purpose of suggesting the design of one way slab, two way slab & both shear and flexural design of beam, using nominal amount of parameter. Here most numbers of parameter in design procedure are taken as a constant value. The main reason behind this is to shorten the time & effort of the designer. Simplified design procedures comply with the provisions of Building Code Requirements for Structural Concrete (ACI 318) using appropriate load factors & strength reduction factors.

The design is formulated in excel & java script which would help the designers & those who are interested in designing the slab & beam in shortest possible time, with minimum amount of effort. Simplified design of other units of structure like columns, footing, stair, shear wall will be carry forward in future.

5.2 Recommendations

1) This paper contains simplified design of reinforced beams and slabs. In order to complete the full building, simplified design of other structural units of building i.e. columns, footings, shear wall etc. have to be formulated.

2) The design can be simplified if the parameters like strength of rebar and concrete can be made constant (i.e. f_y = 60,000 psi, f'_c = 4000 psi).

3) Here the formula $\frac{\text{Peremeter}}{145}$ is used to determine the thickness of slab. But $\frac{\text{Peremeter}}{150}$, $\frac{\text{Peremeter}}{160}$, $\frac{\text{Peremeter}}{180}$ can also be used for determining the slab thickness for economic purpose.

4) Here for design purpose only dead load and live load are considered. But if other loads i.e. wind load and earthquake loads are to be considered, then this loads are taken into account separately in design procedure.

APPENDIX

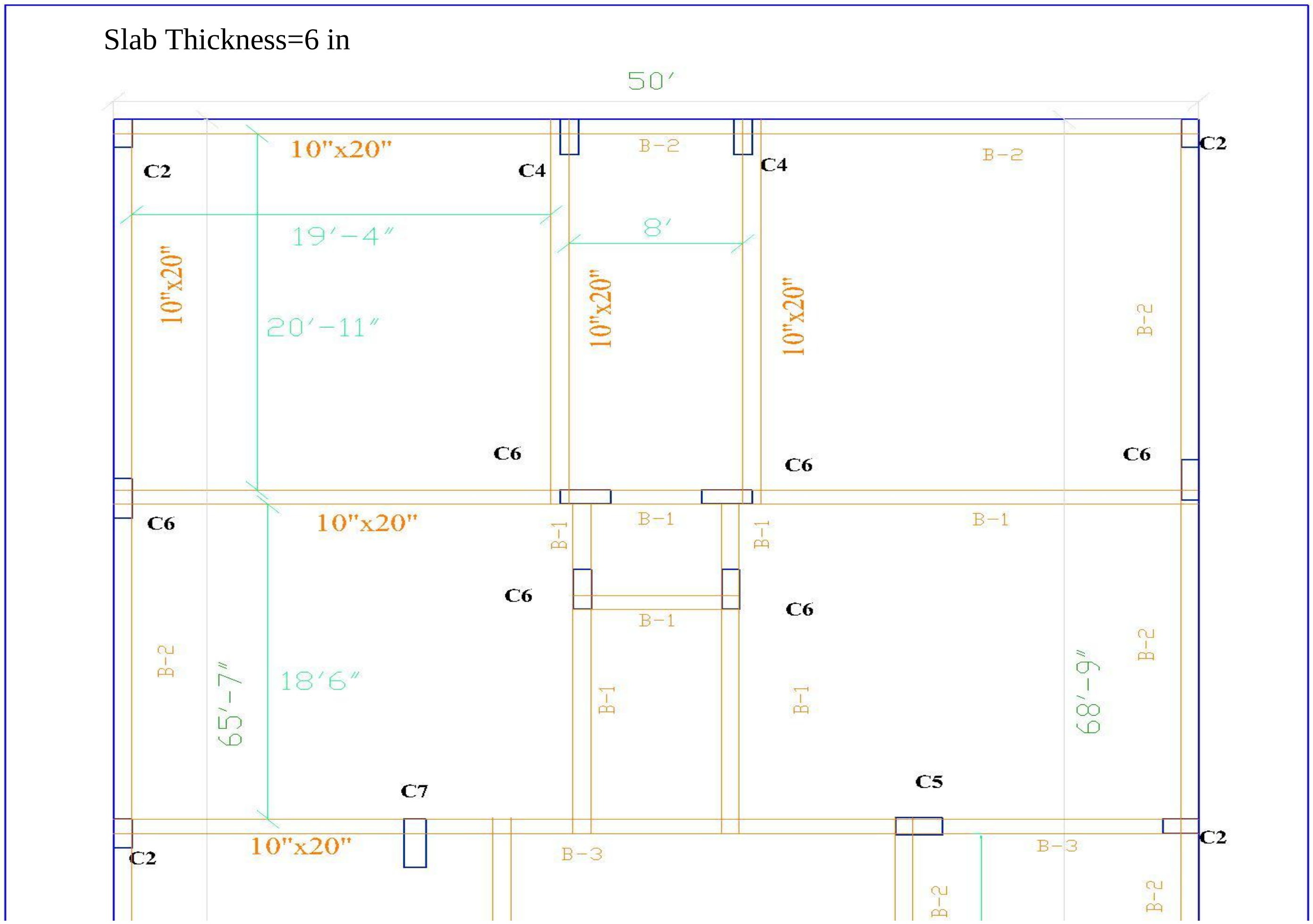


Figure A.01: Partial plan of a slab (for alpha calculation).

Slab Thickness= 5 in

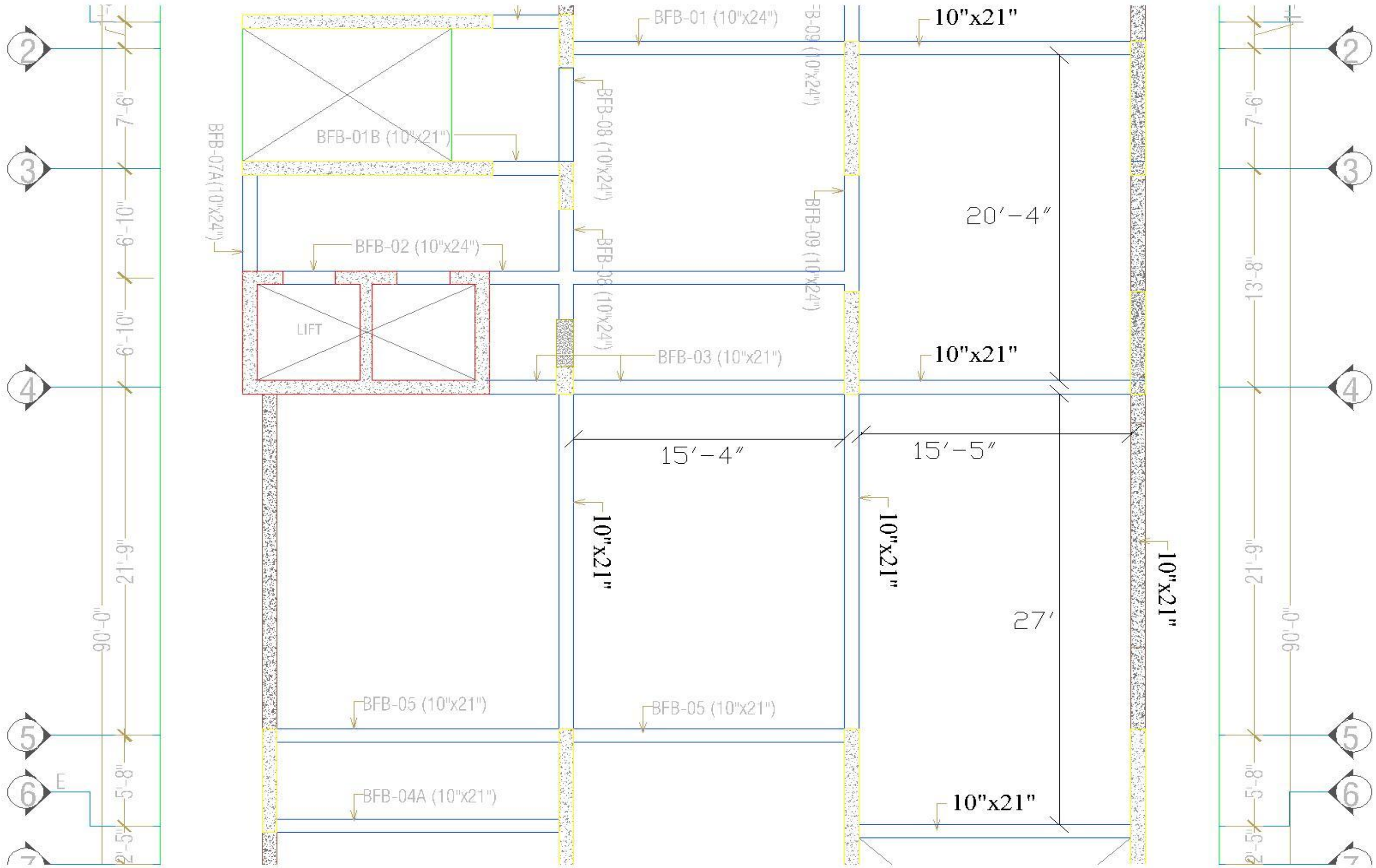


Figure A.02: Partial plan of a slab (for alpha calculation).

Slab Thickness= 5 in

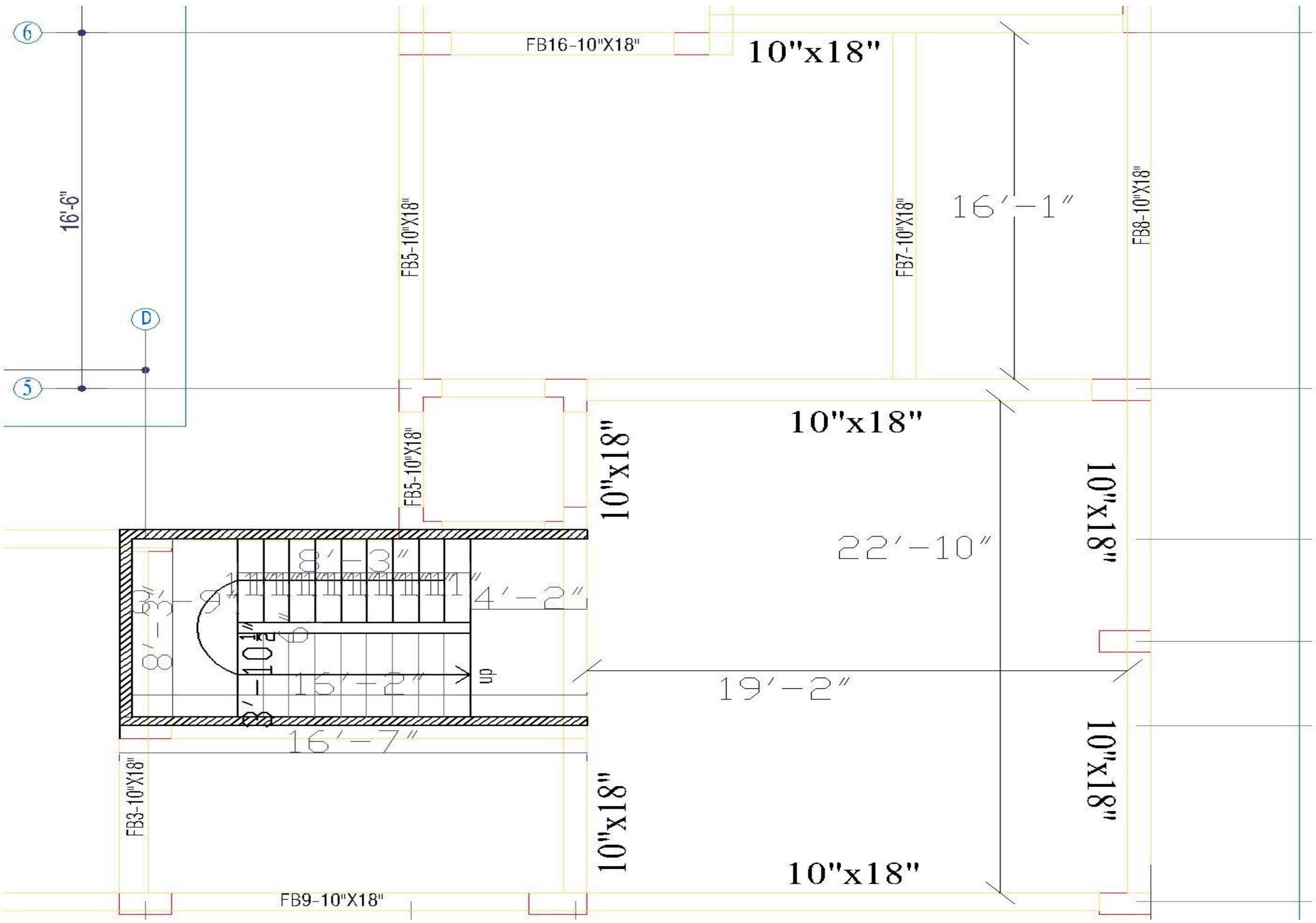


Figure A.03: Partial plan of a slab (for alpha calculation).

Slab Thickness= 6 in

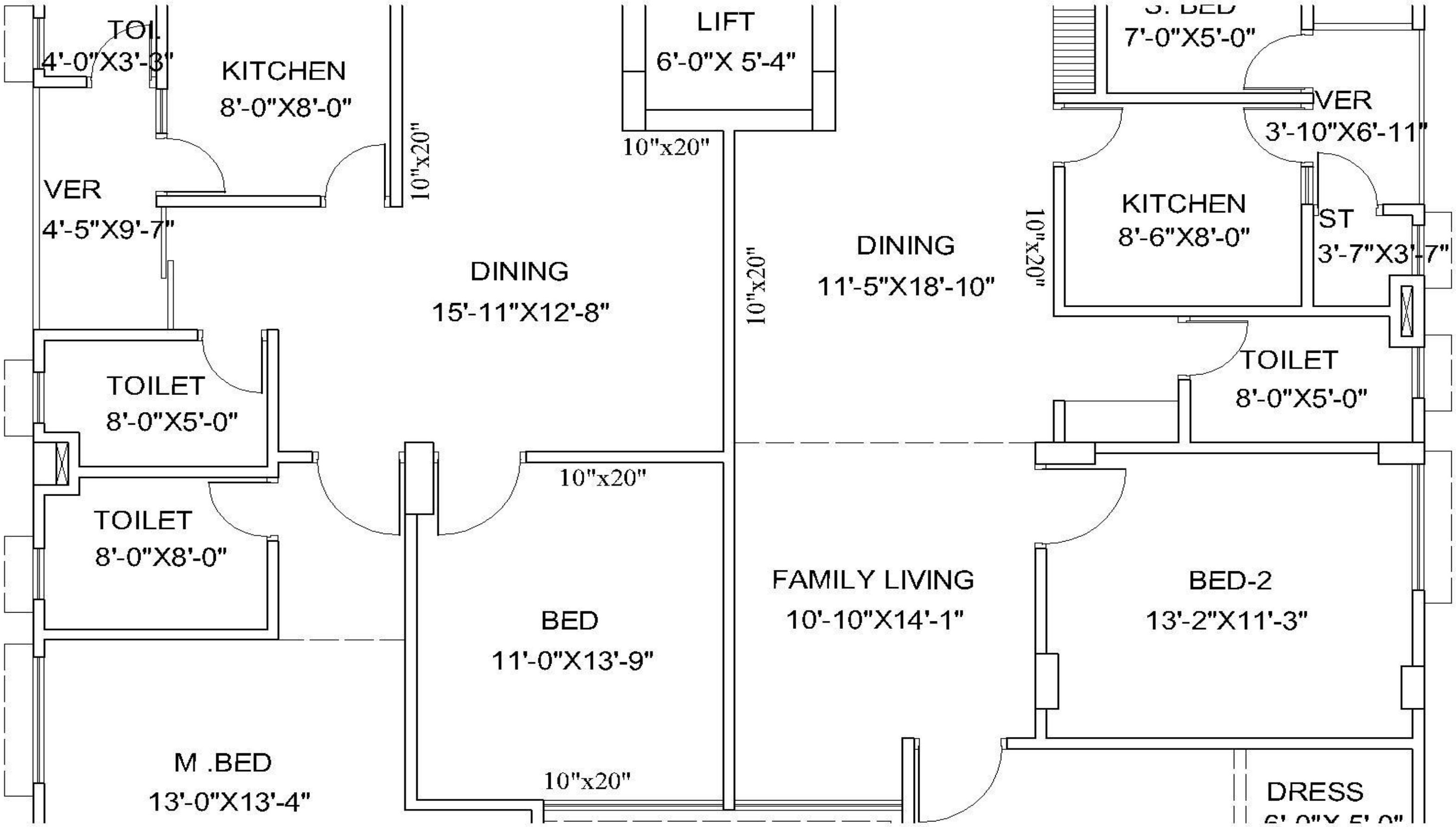


Figure A.04: Partial plan of a slab (for alpha calculation).

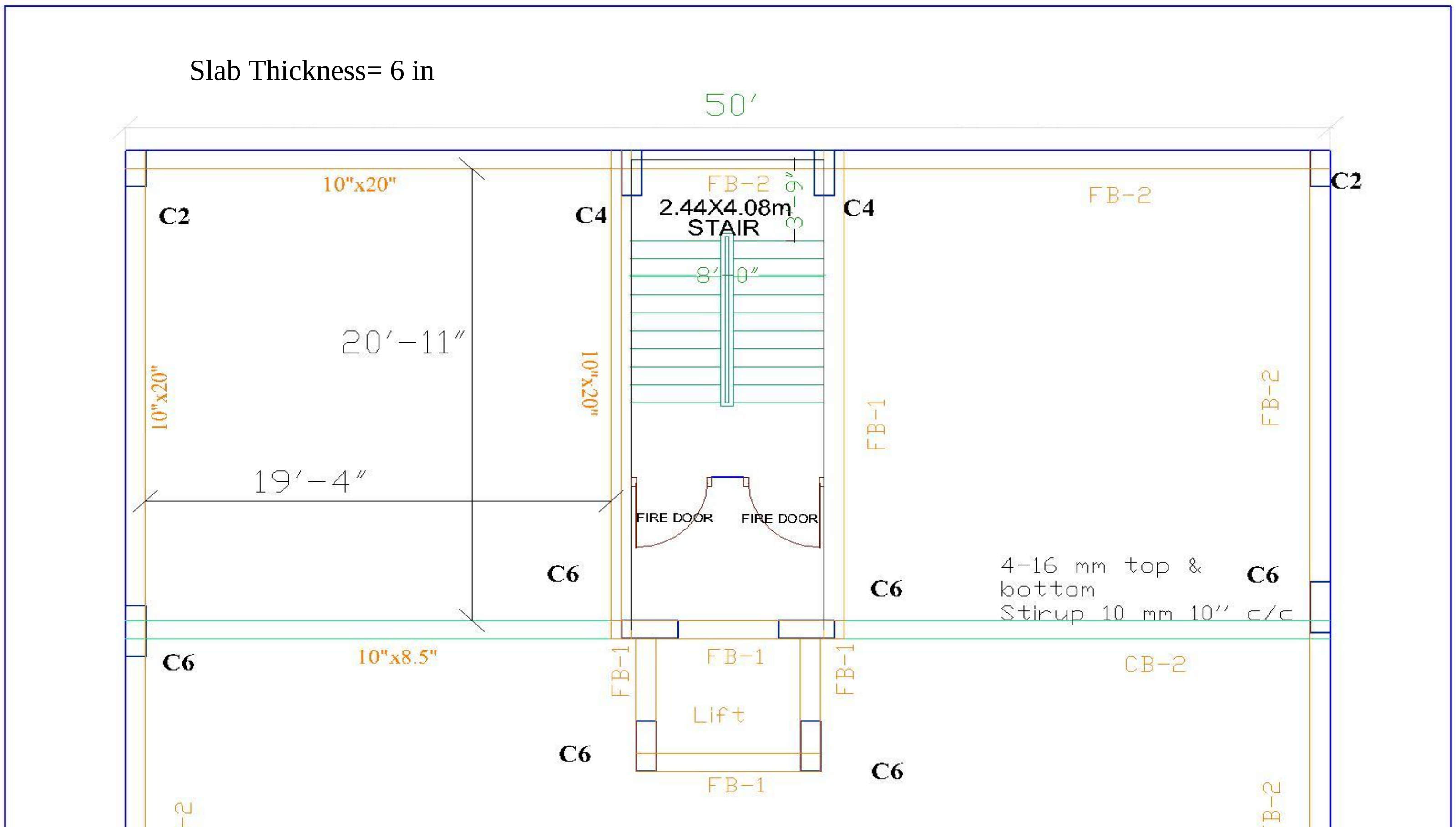


Figure A.05: Partial plan of a slab (for alpha calculation).

Slab Thickness= 6 in

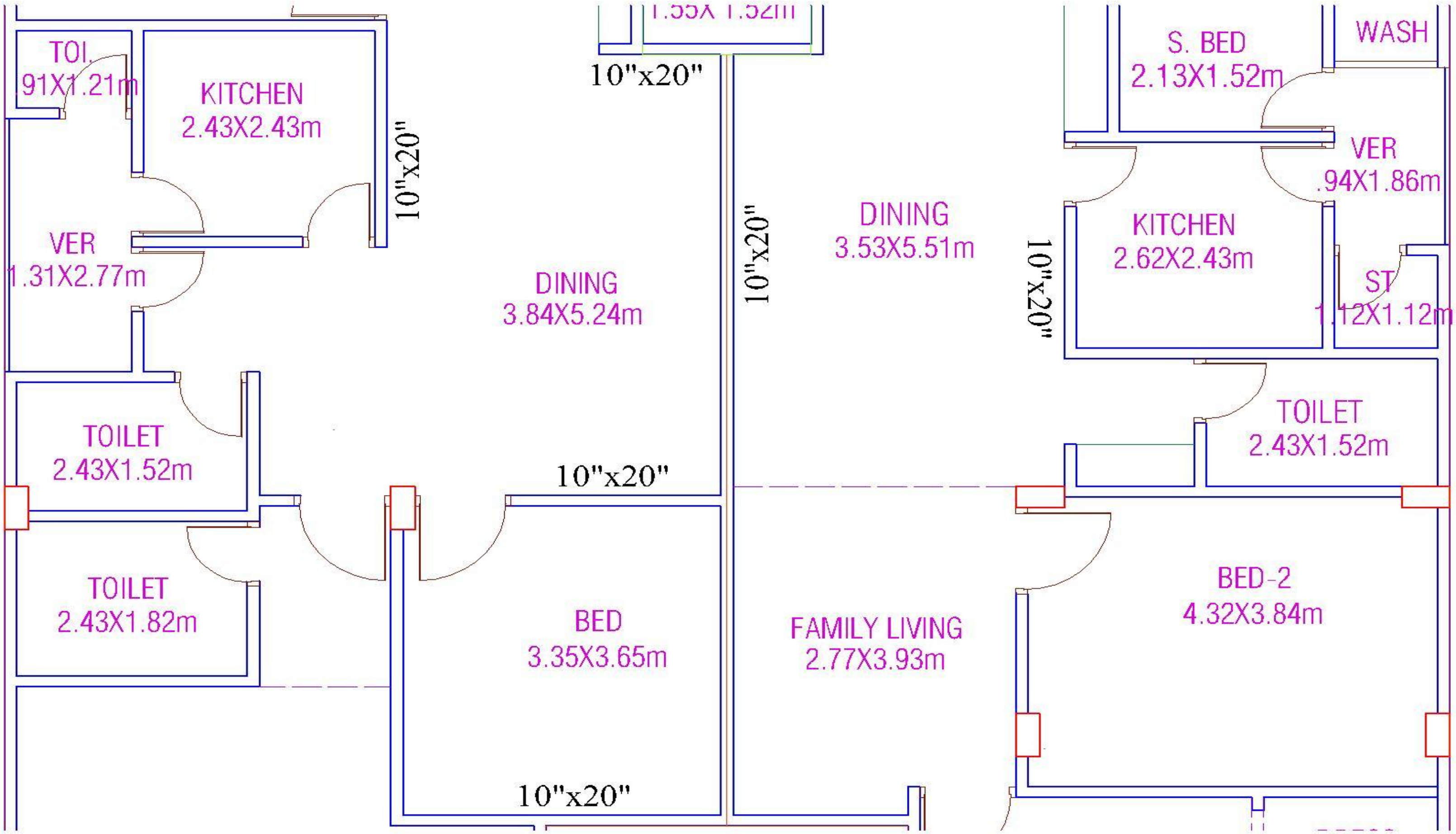


Figure A.06: Partial plan of a slab (for alpha calculation).

Slab Thickness= 6 in

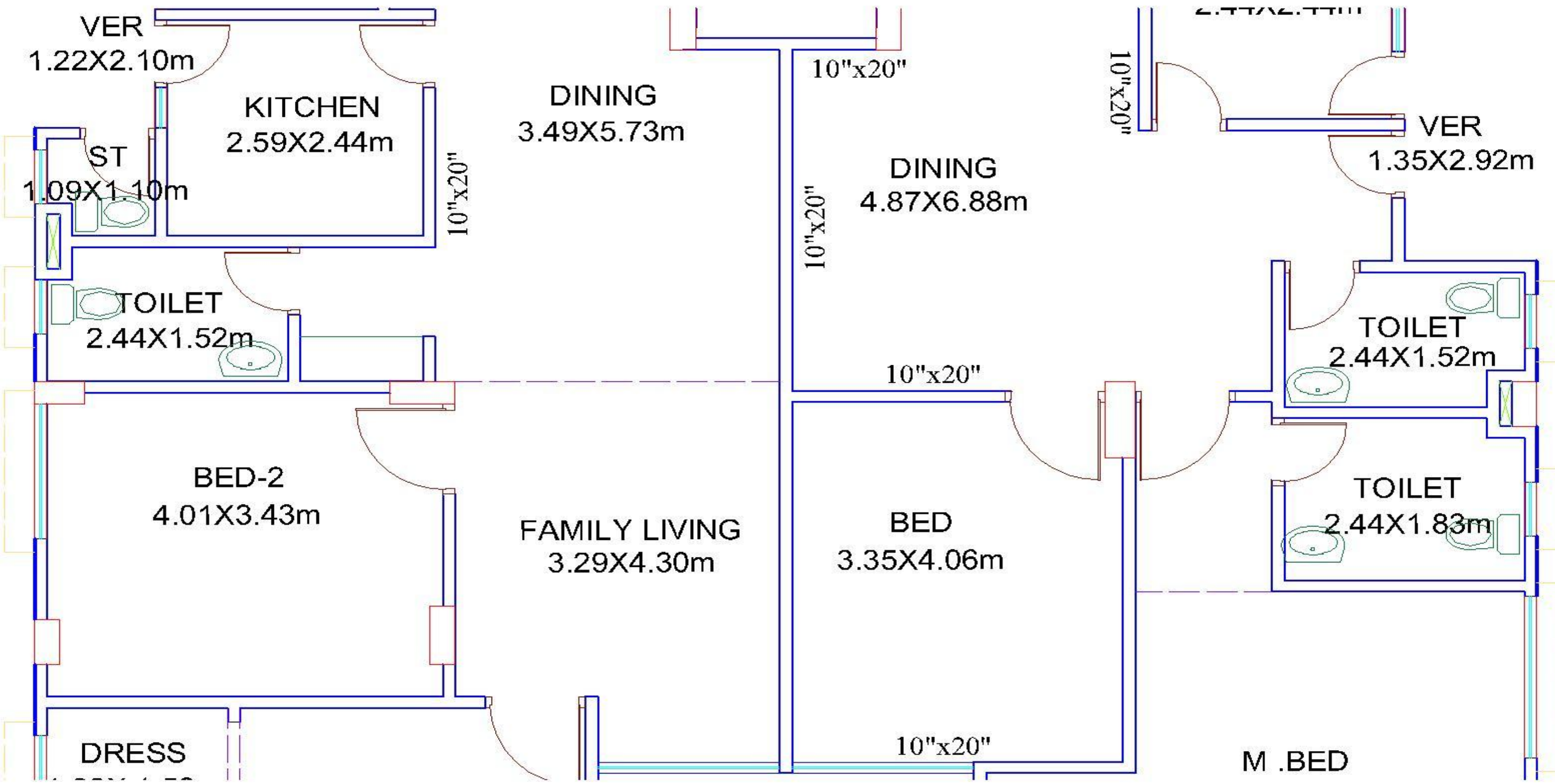


Figure A.07: Partial plan of a slab (for alpha calculation).

Slab Thickness= 6 in

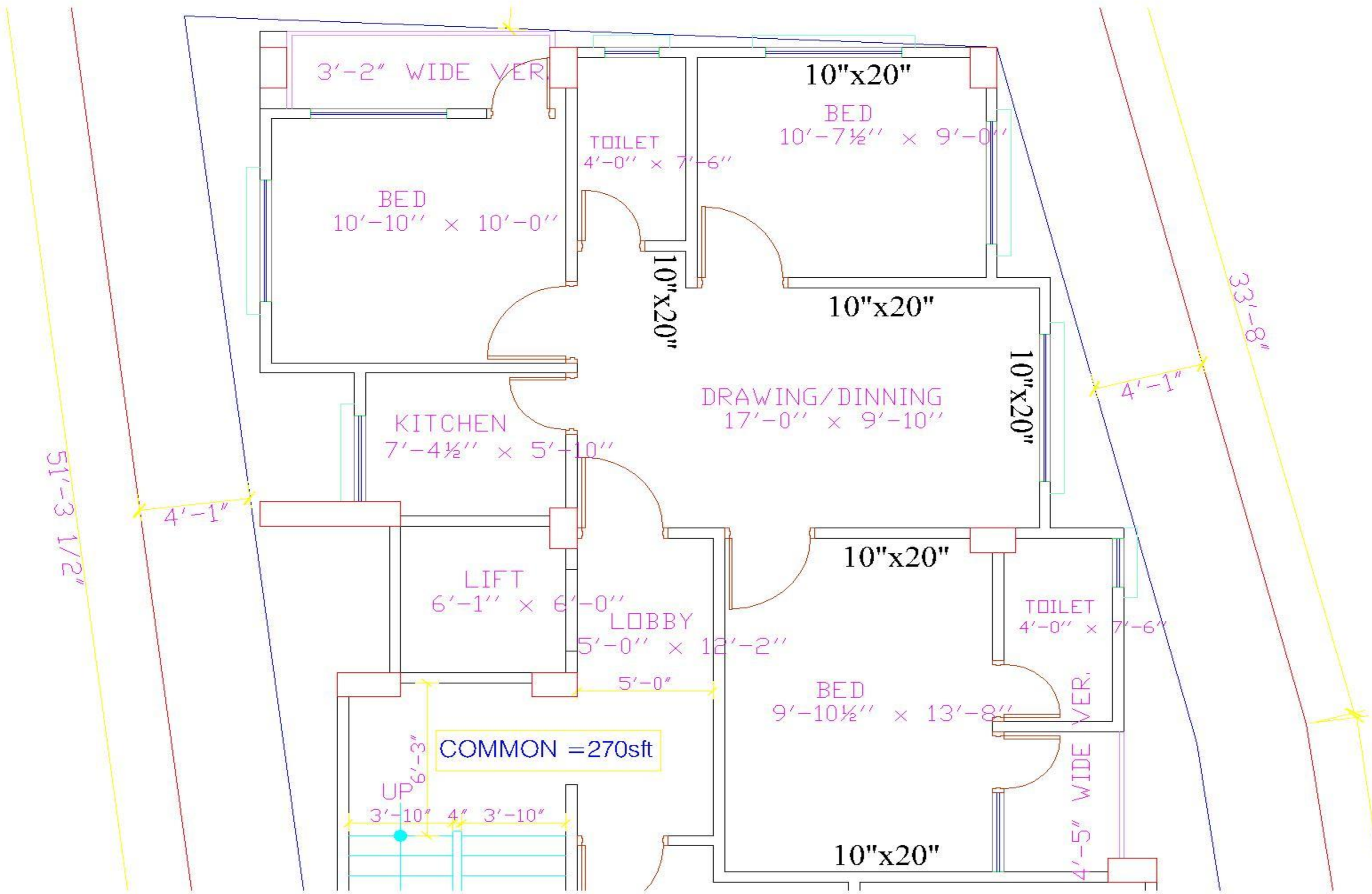


Figure A.08: Partial plan of a slab (for alpha calculation).

Slab Thickness= 6 in

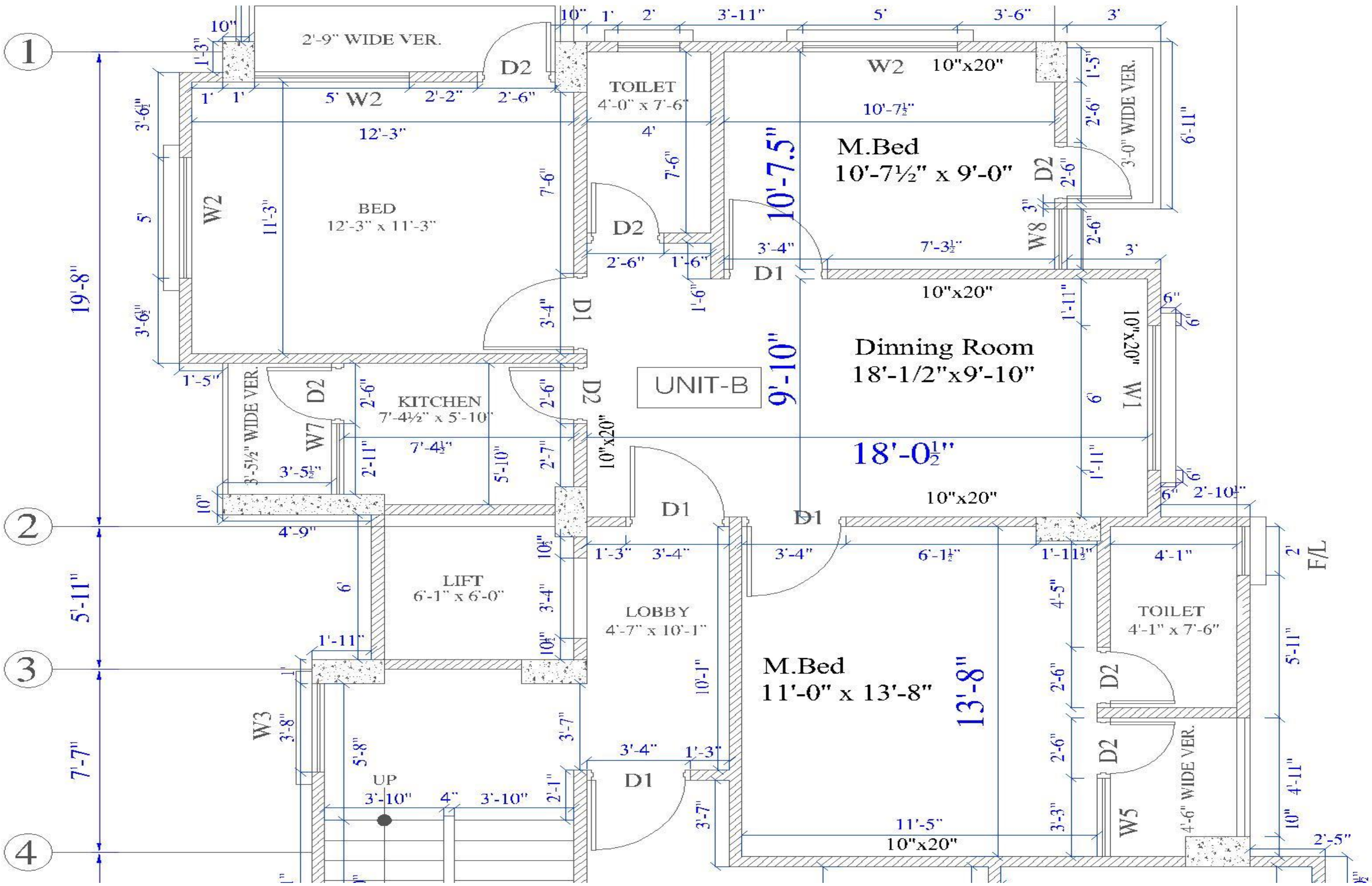


Figure A.09: Partial plan of a slab (for alpha calculation).

Slab Thickness= 5 in

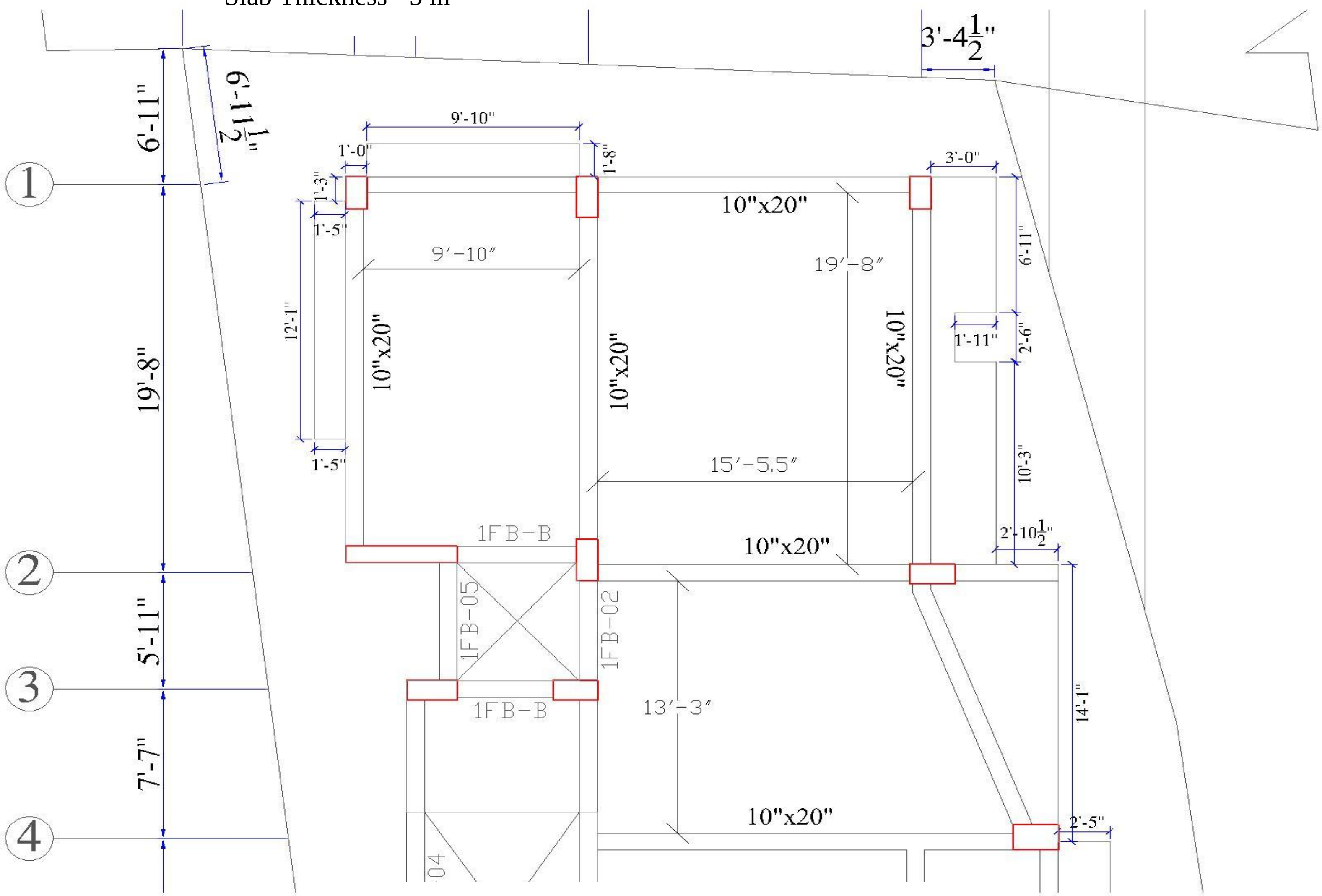


Figure A.10: Partial plan of a slab (for alpha calculation).

Example #03:

Determination of steel area of a one way slab

A reinforced concrete slab is built integrally with its supports and consists of two equal spans, each with a clear span of 15 ft. The service live load is 40 psf and 4000 psi concrete is specified for the use with steel with yield stress equal to 60000 psi.

Solution: According to ACI minimum thickness (Table 2.1):

$l_n/28 = 15\times12/28 = 6.43$ in ≈ 6.50 in.

Self-weight of slab= $150\times6.50/12 = 81.25$ psf (According to ACI, unit weight of concrete is 150 lb/ft^3)

The specified live load and computed dead load are multiplied by the ACI load factors:

Dead load = $81.5\times1.2 = 97.8$ psf

Live load = 40×1.6 = 64 psf

Total load = 161.8 psf

Using ACI moment coefficient (Table 2.2):

At interior support: $-M = 1/9\times0.161\times15^2 = 4.025$ ft-kip

At mid-span: $+M = 1/14\times0.161\times15^2 = 2.5875$ ft-kip

At exterior support: $-M = 1/24\times0.161\times15^2 = 1.51$ ft-kip

Now, $\rho_{max} = 0.85\times\beta\times f'_c/f_y \times \frac{0.003}{0.003+0.004} = (0.85)^2\times4/60\times\frac{0.003}{0.003+0.004} = 0.021$

Minimum required effective depth would be found from ACI equation,

$$d^2 = \frac{M_u}{\phi\rho f_y b(1-0.59\rho\left(\frac{f_y}{f_c'}\right))} = \frac{4.025\times12}{0.9\times0.021\times60\times12\times(1-0.59\times0.021\times\left(\frac{60}{4}\right))} = 4.36\ in^2$$

Now, d = 2.08 in ≈ 2.1 in.

Assumed effective depth= (6.50-1) in= 5.50 in. Which is greater than the required effective depth. So, final slab thickness = 6.50 in.

Assume, stress block depth, a = 1 in.

The area of steel required per foot width in the top of the slab is,

$A_s = \frac{M_u}{\phi f_y \,(d-\left(\frac{a}{2}\right))} = \frac{4.025\times12}{0.9\times60\times(5.5-0.5)} = 0.179\ in^2$

Checking a, $a = \frac{A_s\, f_y}{0.85\, f_c'\, b} = \frac{0.179\times60}{0.85\times4\times12} = 0.26$ in.

Trail 2: a = 0.26 in. So, $A_s = \frac{4.025\times12}{0.9\times60\times(5.5-0.13)} = 0.166\ in^2$

Again checking a, $a = \frac{0.166\times60}{0.85\times4\times12} = 0.245$ in. ≈ 0.26 in. (Ok)

Final A_s, At interior support, $A_s = \frac{4.025\times12}{0.9\times60\times(5.5-0.125)} = 0.166\ in^2/ft$ (Controlling)

At mid span, $A_s = \frac{2.588\times12}{0.9\times60\times(5.5-0.125)} = 0.107\ in^2/ft$ (Controlling $A_{s(min)}$)

At exterior support, $A_s = \frac{1.51\times12}{0.9\times60\times(5.5-0.125)} = 0.06\ in^2/ft$ (Controlling $A_{s(min)}$)

$A_{s(min)} = 0.0018\times b\times t = 0.0018\times12\times6.5 = 0.14\ in^2/ft$

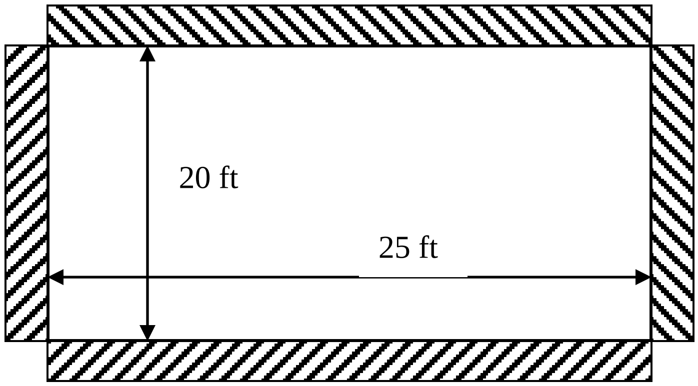
Temperature & shrinkage reinforcement:

According to ACI Code, minimum temperature & shrinkage reinforcement, $A_{s(min)}$ should be provided along the transverse direction of the main reinforcement.

Example #04:

Two way slab design (Interior slab panel)

Solution:



Here, A= 20' and B= 25'= l_n

Thickness, t= 7" (given)

So, d= (7"-1") = 6"

Dead load, W_{DL}= (7+0.5+1.5)*12.5*1.2 = 135 psf

Live load, W_{LL}= 100*1.6 = 160 psf

Total factored load, W_u = 295 psf

m = A/B= 20/25= 0.8

	Case 2
$-C_A$	0.065
$-C_B$	0.027.
$C_{A\,DL}$	0.026
$C_{B\,DL}$	0.011
$C_{A\,LL}$	0.041
$C_{B\,LL}$	0.017

$+M_A= C_{A\,DL}*W_{DL}*A^2+ C_{A\,LL}*W_{LL}*A^2$

$= 0.026*135*20^2+0.041*160*20^2$

$= 4028$ lb-ft/ft

$= 4.208$ k-ft/ft

$-M_A = C_A * W_u * A^2$

$= 0.065*295*20^2$

$= 7670$ lb-ft/ft

$= 7.67$ k-ft/ft

$+M_B = C_{B\ DL} * W_{DL} * B^2 + C_{B\ LL} * W_{LL} * B^2$

$= 0.011*135*25^2 + 0.017*160*25^2$

$= 2628.125$ lb-ft/ft

$= 2.628$ k-ft/ft

$-M_B = C_B * W_u * B^2$

$= 0.027*295*25^2$

$= 4978.125$ lb-ft/ft

$= 4.978$ k-ft/ft

<u>Rebar for short direction/transverse direction:</u>

$+A_{S\ A} = \frac{M*12}{0.9*f_y*\left(d-\frac{a}{2}\right)} = \frac{M*12}{0.9*60*\left(d-\frac{a}{2}\right)} = \frac{M}{4.5*\left(d-\frac{a}{2}\right)} = \frac{4.028}{4.5*(6-\frac{0.319}{2})} = 0.153\ in^2/ft$

and $a = \frac{A_s f_y}{0.85 f_c' b} = \frac{A_s*60}{0.85*3*12} = 0.3$ in

$A_{s\ min} = 0.0018*b*t*1.5 = 0.0018*12*7*1.5 = 0.2268\ in^2/ft$ (Controlling)

Using Φ 10 mm bar

$S = \frac{\text{area of bar used} * \text{width of strip}}{\text{requried } A_s} = \frac{0.121*12}{0.2268}$ = 6.4" ≈ 6" c/c at bottom along short direction

crank 50% bar to negative zone.

$-A_{S\ A} = \frac{M}{4.5*\left(d-\frac{a}{2}\right)} = \frac{7.67}{4.5*(6-0.60/2)}$ = 0.3 in^2/ft (Controlling)

$A_{s\ min}$ = 0.2268 in^2/ft

Already provided, $A_{s1} = \frac{0.121*12}{12}$ = 0.121 in^2/ft

Extra top required, A_{s2} = (0.3-0.121) = 0.179 in^2/ft

Using Φ10 mm bar, S = 8.11" ≈ 8" c/c extra top.

<u>Rebar along long direction:</u>

$+A_{S\ B} = \frac{2.628}{4.5*(6-0.2082/2)}$ = 0.1 in^2/ft

$A_{s\ min}$ = 0.2268 in^2/ft (Controlling)

Using Φ10 mm bar @ 6.4" ≈ 6" c/c at bottom along long direction crank 50% bar to negative zone.

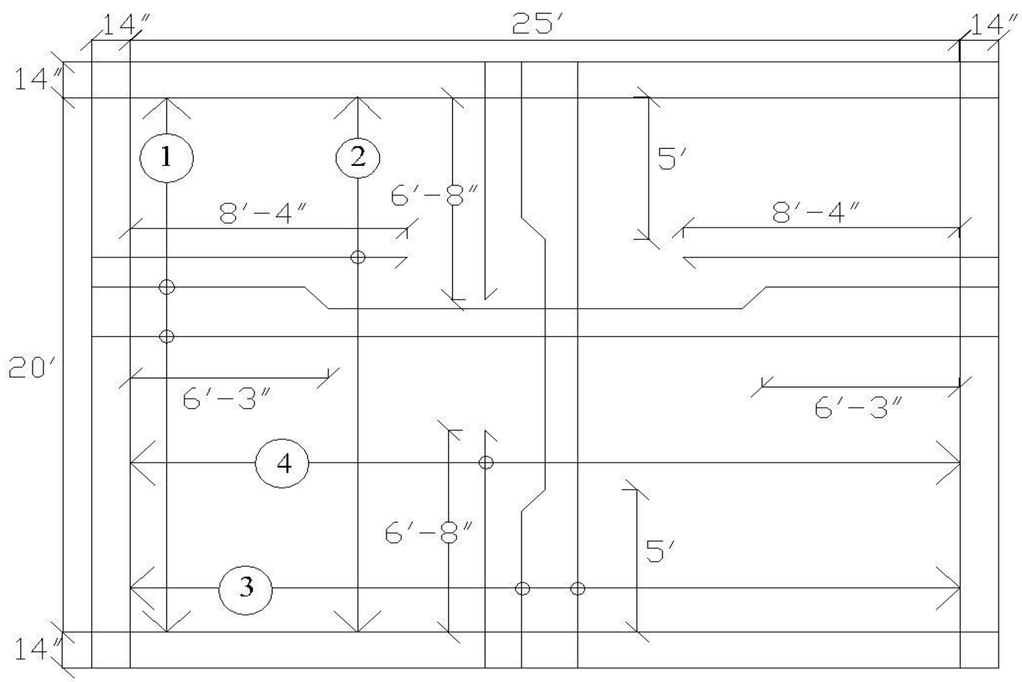
$-A_{S\ B} = \frac{4.978}{4.5*(6-0.394/2)}$ = 0.19 in^2/ft

$A_{s\ min}$ = 0.2268 in^2/ft (Controlling)

Already provided, $A_{s1} = \frac{0.121*12}{12}$ = 0.121 in^2/ft

Extra top required, A_{s2} = (0.2268-.121) in^2/ft = 0.2198 in^2/ft

Using Φ10 mm bar @ 13.72" ≈ 13.5" c/c extra top.



Legends:

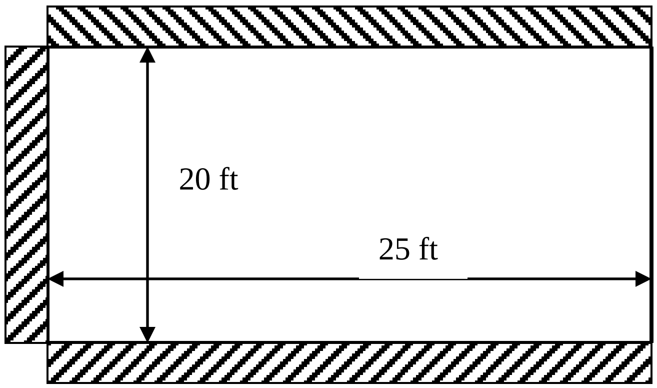
(**1**) Ø 10 mm @ 6" c/c althrough cranked alternatively.
(**2**) Ø 10 mm @ 13.5" c/c extra top.
(**3**) Ø 10 mm @ 6" c/c althrough cranked alternatively.
(**4**) Ø 10 mm @ 8" c/c extra top.
All beams are 14"×14"
Slab Thickness= 7"

Figure A.11: Reinforcement details of a slab (Example #04).

Example #05:

Two way slab design (Exterior slab panel)

Solution:



Here, A= 20' and B= 25'= l_n.

Thickness, t= 7" (given)

So, d= (7"-1") = 6"

Dead load, W_{DL}= (7+0.5+1.5)*12.5*1.2 = 135 psf

Live load, W_{LL}= 100*1.6 = 160 psf

Total factored load, W_u = 295 psf

m = A/B= 20/25= 0.8

	Case 9
$-C_A$	0.075
$-C_B$	0.017
$C_{A\ DL}$	0.029
$C_{B\ DL}$	0.010
$C_{A\ LL}$	0.042
$C_{B\ LL}$	0.017

$+M_A = C_{A\ DL}*W_{DL}*A^2 + C_{A\ LL}*W_{LL}*A^2$

$= 0.029*135*20^2+0.042*160*20^2$

= 4254 lb-ft/ft

= 4.254 k-ft/ft

$-M_A = C_A * W_u * A^2$

$= 0.075*295*20^2$

= 8850 lb-ft/ft

= 8.85 k-ft/ft

$+M_B = C_{B\,DL} * W_{DL} * B^2 + C_{B\,LL} * W_{LL} * B^2$

$= 0.010*135*25^2 + 0.017*160*25^2$

= 2543.75 lb-ft/ft

= 2.543 k-ft/ft

$-M_B = C_B * W_u * B^2$

$= 0.017*295*25^2$

= 3134.75 lb-ft/ft

= 3.164 k-ft/ft

<u>Rebar for short direction/transverse direction:</u>

$+A_{S\,A} = \frac{M*12}{0.9*f_y*\left(d-\frac{a}{2}\right)} = \frac{M*12}{0.9*60*\left(d-\frac{a}{2}\right)} = \frac{M}{4.5*\left(d-\frac{a}{2}\right)} = \frac{8.85}{4.5*(6-\frac{0.7}{2})} = 0.1673\ in^2/ft$

and $a = \frac{A_s f_y}{0.85 f_c' b} = \frac{A_s*60}{0.85*3*12} = 0.7$ in

$A_{s\,min} = 0.0018*b*t*1.5 = 0.0018*12*7*1.5 = 0.2268\ in^2/ft$ (Controlling)

Using Φ 10 mm bar

$S = \frac{\text{area of bar used} * \text{width of strip}}{\text{requried} \ A_s} = \frac{0.121*12}{0.2268}$ = 6.4" ≈ 6"c/c at bottom along short direction

crank 50% bar to negative zone.

$-A_{S\ A} = \frac{M}{4.5*\left(d-\frac{a}{2}\right)} = \frac{7.67}{4.5*(6-0.6/2)}$ = 0.3in^2/ft (Controlling)

$A_{s\ min}$ = 0.2268 in^2/ft

Already provided, $A_{s1} = \frac{0.121*12}{12}$ = 0.121 in^2/ft

Extra top required, A_{s2} = (0.3-0.121) = 0.179 in^2/ft

Using Φ10 mm bar, S= 8.11" ≈ 8" c/c extra top.

<u>Rebar along long direction:</u>

$+A_{S\ B} = \frac{2.543}{4.5*(6-0.2/2)}$ = 0.095 in^2/ft

$A_{s\ min}$ = 0.2268 in^2/ft (Controlling)

Using Φ10 mm bar @ 6.4"≈ 6" c/c at bottom along long direction crank 50% bar to negative zone.

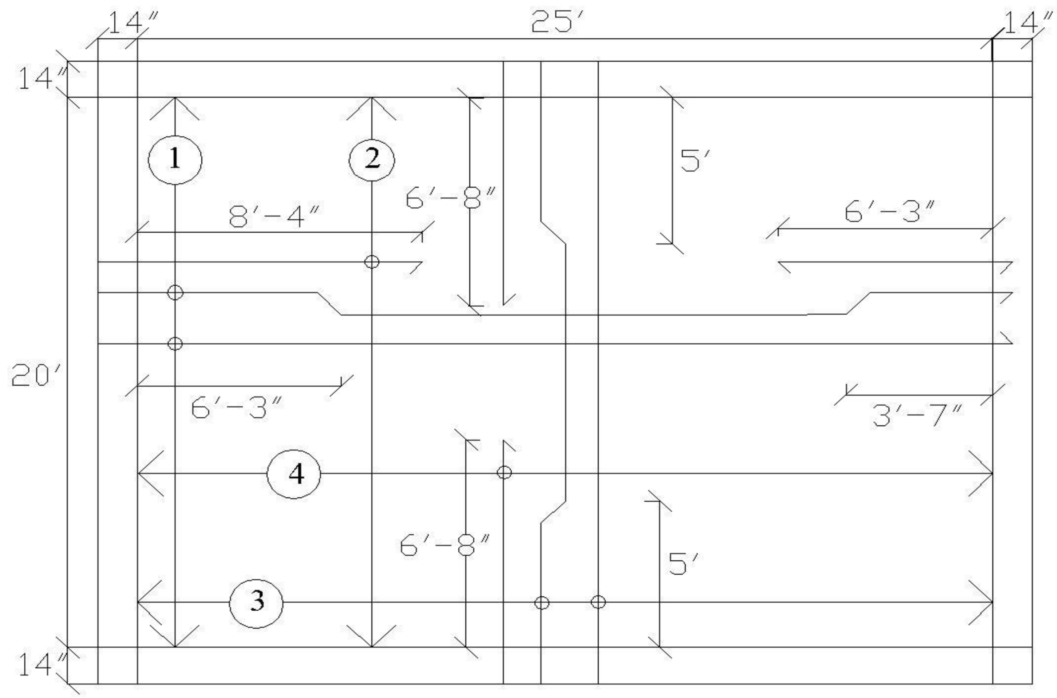
$-A_{S\ B} = \frac{3.134}{4.5*(6-0.248/2)}$ = 0.118 in^2/ft

$A_{s\ min}$ = 0.2268 in^2/ft. (Controlling)

Already provided, $A_{s1} = \frac{0.121*12}{12}$ = 0.121 in^2/ft

Extra top required, A_{s2} = (0.2268-.121) in^2/ft = 0.2198 in^2/ft

Using Φ10 mm bar @ 13.72" ≈ 13.5" c/c extra top.



Legends:

(**1**) Ø 10 mm @ 6" c/c althrough cranked alternatively.
(**2**) Ø 10 mm @ 13.5" c/c extra top.
(**3**) Ø 10 mm @ 6" c/c althrough cranked alternatively.
(**4**) Ø 10 mm @ 8" c/c extra top.
All beams are 14"×14"
Slab Thickness= 7"

Figure A.12: Reinforcement details of a slab (Example #05).

Example #06:

Simplified design of a beam (Flexure design)

Design an interior beam of 16 ft long. Width of beam is 12 in. Slab thickness is 6 in. Clear span of beam along right and left side is 16 ft and 14 ft respectively. The beam is one end continuous. Live load is 40 psf. No partition wall. Floor finish load is 30 psf given. If f'_c = 3,000 psi and f_y = 60,000 psi, design the beam for flexure only.

Solution: According to ACI Code (Table 2.7) minimum thickness of beam:

$t_{min} = \frac{l}{18.5}$ Where, l= length of the beam.

$= \frac{16*12}{18.5}$ = 10.378 in ≈ 10.5 in (say).

So, effective depth of the beam, d_{given}= (10.5-2.5) in = 8 in

Self weight of the slab = (16/2+14/2)*(6/12)*0.15 k/ft = 1.125 k/ft

Live load = (15*0.04) + (12/12*0.04) k/ft = 0.64 k/ft

Self weight of beam = $\frac{10.5*12}{144} * 0.15$ k/ft = 0.13125 k/ft

Floor finish = (8+7+$\frac{12}{12}$)*0.03 k/ft = 0.48 k/ft

Total load = (1.2*dead load)+(1.6*live load)

= 1.2×(1.125+0.13125+0.48)+1.6*0.64 k/ft = 3.1075 k/ft

Maximum positive moment, +M = $\frac{wl^2}{14}$ = $(3.1075*16^2)/14$ = 56.823 k-ft

Maximum negative moment, -M = $\frac{wl^2}{9}$ = $(3.1075*16^2)/9$ = 88.391 k-ft

Here, $\rho = 0.85*\beta_1*(f'_c/f_y)*\{\varepsilon_u/(\varepsilon_u+\varepsilon_t)\}$ = 0.85*0.85*(4/60)*{0.003/(0.003+0.004)}

= 0.015482

b will be the smallest of,

$b \leq (16/4)*12 = 48$ in, or (Selecting)

$\leq 16\ h_f + b_w = 108$ in, or

$\leq$ (c/c distance/2) = 90 in.

Now, $R_n = \rho\ f_y\ (1- \frac{0.5\rho f_y}{0.85f'_c}) = 759.73$ psi

Again, $(bd^2)_{req} = \frac{M_u}{\emptyset R_n}$

So, $d_{req} = \sqrt{\frac{M_u}{\emptyset b R_n}} = 5.68$ in. Here, $d_{provided} > d_{req}$ (Ok).

$+A_s = \frac{M}{\emptyset f_y (d - \frac{a}{2})} = \frac{56.823*12*1000}{0.9*60000*(8-\frac{0.81}{2})} = 1.66\ in^2$ (Controlling)

And, $a = \frac{A_s f_y}{0.85 f'_c b} = \frac{1.66*60}{0.85*3*48} = 0.81$ in

Use 2 Ø 28 mm cont. $A_{s\ given} = 2*0.95 = 1.9\ in^2$

Again, $- A_s = \frac{M}{\emptyset f_y (d - \frac{a}{2})} = \frac{88.391*12*1000}{0.9*60000*(8-\frac{1.27}{2})} = 2.67\ in^2$ (Controlling)

And, $a = \frac{A_s f_y}{0.85 f'_c b} = \frac{2.6*60}{0.85*3*48} = 1.27$ in

Use 2 Ø 28 mm cont. + 2 Ø 20 mm extra over support. $A_{s\ given} = 2*0.95+2*0.48= 2.86\ in^2$

(Continuing 70% of steel area)

$A_{s\ min} = \frac{200}{f_y} \times bd = \frac{200}{60000} \times 12 \times 8 = 0.32\ in^2$

(For reinforcement details see Figure 2.9)

Example #07:

Simplified design of a beam (Shear design)

Solution: The example illustrates the simple procedure for selecting stirrups design values for V_c and V_s.

(1) Design data: f'_c = 4,000 psi, f_y = 60,000 psi, W_u = 7 kips/ft.

(2) Beam dimensions: b_w = 12 in, d = 24 in.

(3) Calculations:

V_u @ column centerline:	W_u*l/2= 7*24/2= 84.0 kips
V_u @ face of support:	84 -1.17(7)= 75.8 kips
V_u @ d from support face (critical section):	75.8-2(7)= 61.8 kips
$(\O V_c+\O V_s)_{max}$:	$0.48b_wd$= 0.48(12)(24)= 138.2 kips
$\O V_c$:	$0.095b_wd$= 0.095(12)(24)= 27.4 kips
$\O V_c/2$:	$0.048b_wd$= 0.048(12)(24)= 13.80 kips

(The calculations come from ACI Code (Table 2.11))

Since 138.2 kips > 61.8 kips, beam size is adequate for shear strength.

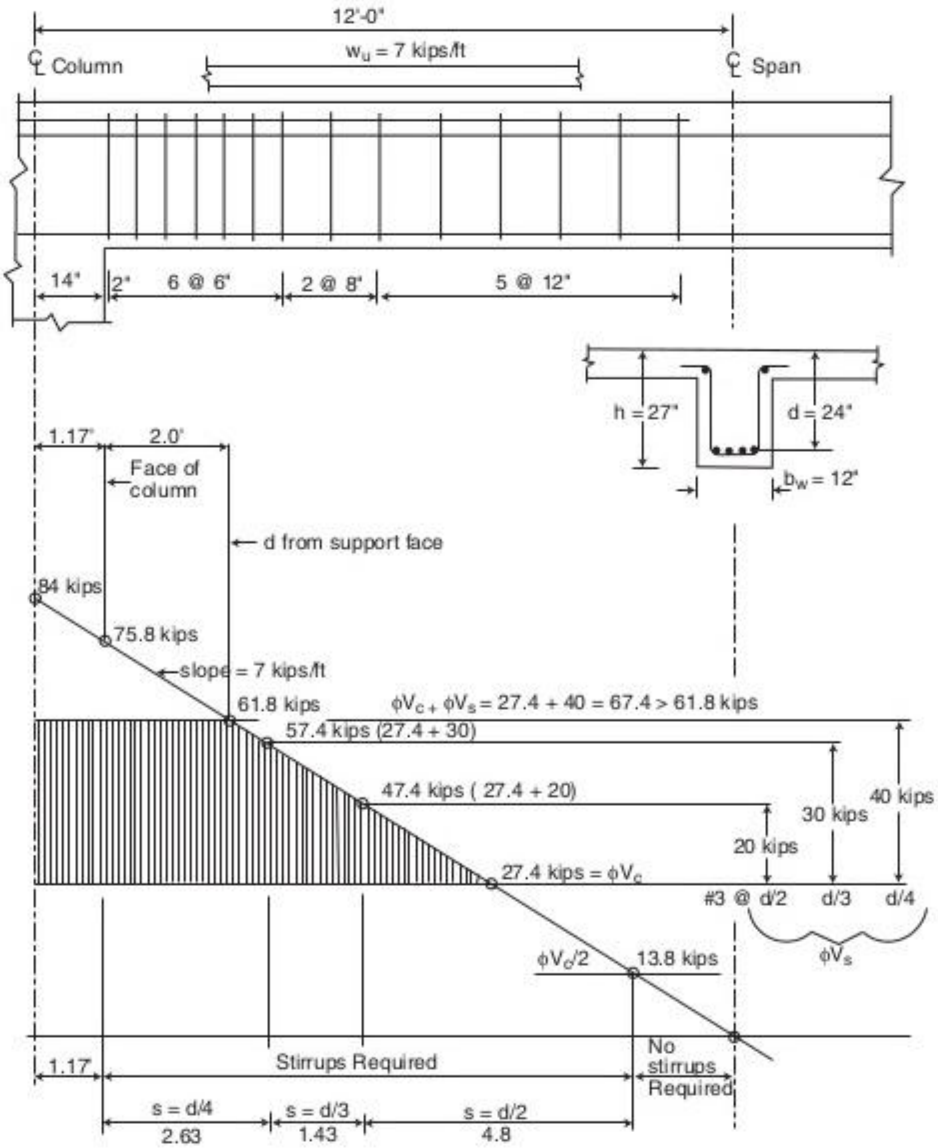


Figure A.13: Simplified method for stirrup spacing (Example #07).

$ØV_s$ for No.3 stirrups at d/2, d/3, and d/4 are scaled vertically from $ØV_{c.}$ The horizontal intersection of the $ØV_s$ values (20 kips, 30 kips, and 40 kips) with the shear diagram automatically sets the distances where the No.3 stirrups should be spaced at d/2, d/3, and d/4. The exact numerical values for those horizontal distances are calculated as follows (although scaling from the sketch is close enough for practical design):

No.3 @ d/4 = 6 in.: (75.8-57.4)/7 = 2.63 ft (31.5 in.) use 6 @ 6 in.

@ d/3 = 8 in.: (57.4-47.4)/7 = 1.43 ft (17.0 in.) use 2 @ 8 in.

@ d/2 = 12 in.: (47.4-13.8)/7 = 4.8 ft (57.6 in.) use 5 @ 12 in.

A more practical solution may be to eliminate the 2 @ 8 in. and use 9 @ 6 in. and 5 @ 12 in.

Table A.01 (a): Design loads for various occupancy (According to ASCE).[A.1]

Occupancy or Use	Uniform psf (kN/m^2)	Conc. lbs (kN)
Apartments (see residential)		
Access floor systems		
Office use	50 (2.4)	2000 (8.9)
Computer use	100 (4.79)	2000 (8.9)
Armories and drill rooms	150 (7.18)	
Assembly areas and theaters		
Fixed seats (fastened to floor)	60 (2.87)	
Lobbies	100 (4.79)	
Movable seats	100 (4.79)	
Platforms (assembly)	100 (4.79)	
Stage floors	150 (7.18)	
Balconies (exterior)	100 (4.79)	
On one- and two-family residences only, and not exceeding 100 $ft.^2$ (9.3 m^2)	60 (2.87)	
Bowling alleys, poolrooms, and similar recreational areas	75 (3.59)	
Catwalks for maintenance access	40 (1.92)	300 (1.33)
Corridors		
First floor	100 (4.79)	
Other floors, same as occupancy served except as indicated		
Dance halls and ballrooms	100 (4.79)	
Decks (patio and roof)		
Same as area served, or for the type of occupancy accommodated		
Dining rooms and restaurants	100 (4.79)	
Dwellings (see residential)		
Elevator machine room grating (on area of 4 $in.^2$ (2580 mm^2))		300 (1.33)
Finish light floor plate construction (on area of 1 $in.^2$ (645 mm^2))		200 (0.89)
Fire escapes	100 (4.79)	
On single-family dwellings only	40 (1.92)	
Fixed ladders		See Section 4.4
Garages (passenger vehicles only)	40 (1.92)	Note (1)
Trucks and buses	Note (2)	

(continued)

Table A.01 (b): Design loads for various occupancy (According to ASCE).

Occupancy or Use	Uniform psf (kN/m^2)	Conc. lbs (kN)
Grandstands (see stadium and arena bleachers)		
Gymnasiums, main floors, and balconies	100 (4.79) Note (4)	
Handrails, guardrails, and grab bars	See Section 4.4	
Hospitals		
Operating rooms, laboratories	60 (2.87)	1000 (4.45)
Private rooms	40 (1.92)	1000 (4.45)
Wards	40 (1.92)	1000 (4.45)
Corridors above first floor	80 (3.83)	1000 (4.45)
Hotels (see residential)		
Libraries		
Reading rooms	60 (2.87)	1000 (4.45)
Stack rooms	150 (7.18) Note (3)	1000 (4.45)
Corridors above first floor	80 (3.83)	1000 (4.45)
Manufacturing		
Light	125 (6.00)	2000 (8.90)
Heavy	250 (11.97)	3000 (13.40)
Marquees and canopies	75 (3.59)	
Office buildings		
File and computer rooms shall be designed for heavier loads based on anticipated occupancy		
Lobbies and first floor corridors	100 (4.79)	2000 (8.90)
Offices	50 (2.40)	2000 (8.90)
Corridors above first floor	80 (3.83)	2000 (8.90)
Penal institutions		
Cell blocks	40 (1.92)	
Corridors	100 (4.79)	
Residential		
Dwellings (one- and two-family)		
Uninhabitable attics without storage	10 (0.48)	
Uninhabitable attics with storage	20 (0.96)	
Habitable attics and sleeping areas	30 (1.44)	
All other areas except stairs and balconies	40 (1.92)	
Hotels and multifamily houses		
Private rooms and corridors serving them	40 (1.92)	
Public rooms and corridors serving them	100 (4.79)	
Reviewing stands, grandstands, and bleachers	100 (4.79) Note (4)	
Roofs	See Sections 4.3 and 4.9	

(*continued*)

Table A.01 (c): Design loads for various occupancy (According to ASCE).

Occupancy or Use	Uniform psf (kN/m^2)	Conc. lbs (kN)
Schools		
Classrooms	40 (1.92)	1000 (4.45)
Corridors above first floor	80 (3.83)	1000 (4.45)
First floor corridors	100 (4.79)	1000 (4.45)
Scuttles, skylight ribs, and accessible ceilings		200 (9.58)
Sidewalks, vehicular driveways, and yards subject to trucking	250 (11.97) Note (5)	8000 (35.60) Note (6)
Stadiums and arenas		
Bleachers	100 (4.79) Note (4)	
Fixed Seats (fastened to floor)	60 (2.87) Note (4)	
Stairs and exit-ways	100 (4.79)	Note (7)
One- and two-family residences only	40 (1.92)	
Storage areas above ceilings	20 (0.96)	
Storage warehouses (shall be designed for heavier loads if required for anticipated storage)		
Light	125 (6.00)	
Heavy	250 (11.97)	
Stores		
Retail		
First floor	100 (4.79)	1000 (4.45)
Upper floors	75 (3.59)	1000 (4.45)
Wholesale, all floors	125 (6.00)	1000 (4.45)
Vehicle barriers	See Section 4.4	
Walkways and elevated platforms (other than exit-ways)	60 (2.87)	
Yards and terraces, pedestrians	100 (4.79)	

Notes

(1) Floors in garages or portions of building used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of Table 4-1 or the following concentrated load: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, 3000 lb (13.35 kN) acting on an area of 4.5 in. by 4.5 in. (114 mm by 114 mm, footprint of a jack); (2) for mechanical parking structures without slab or deck which are used for storing passenger car only, 2250 lb (10 kN) per wheel.

(2) Garages accommodating trucks and buses shall be designed in accordance with an approved method, which contains provisions for truck and bus loadings.

(3) The loading applies to stack room floors that support nonmobile, double-faced library bookstacks subject to the following limitations:

a. The nominal bookstack unit height shall not exceed 90 in. (2290 mm);

b. The nominal shelf depth shall not exceed 12 in. (305 mm) for each face; and

c. Parallel rows of double-faced bookstacks shall be separated by aisles not less than 36 in. (914 mm) wide.

(4) In addition to the vertical live loads, the design shall include horizontal swaying forces applied to each row of the seats as follows: 24 lbs/ linear ft of seat applied in a direction parallel to each row of seats and 10 lbs/ linear ft of seat applied in a direction perpendicular to each row of seats. The parallel and perpendicular horizontal swaying forces need not be applied simultaneously.

(5) Other uniform loads in accordance with an approved method, which contains provisions for truck loadings, shall also be considered where appropriate.

(6) The concentrated wheel load shall be applied on an area of 4.5 in. by 4.5 in. (114 mm by 114 mm, footprint of a jack).

(7) Minimum concentrated load on stair treads (on area of 4 in.^{2} (2580 mm^2)) is 300 lbs (1.33 kN).

Table A.02: Design loads for various occupancy (According to BNBC).[A.2]

BUILDING			w [1]	P [2]
Occupancy		Use of floor	kN/m²	kN
RESIDENTIAL (Occupancy - A)	One-or two-family dwellings	1 Room, internal corridor, private stair	2.0	1.8
		2 External stair and corridor	3.0	2.7
	Hotel, hostel, boarding house, flat and apartment, bungalow	1 Bed room, living room, bath room, toilet, dressing room	2.0	1.8
		2 Office room	2.5	2.7
		3 Cafeteria, restaurant, kitchen, laundry, lobby, lounge, game room, dining hall, balcony.	3.0	4.5
		4 Corridor, retail store, staircase	4.0	4.5
		5 Store room	5.0	4.5
		6 Garage, car parking floor, ramp (See Occupancy - K)	-	-
EDUCATIONAL, INSTITUTIONAL & HEALTH-CARE (Occupancy - B, C & D)	School, college, university, penal and mental institutions, orphanage, child care home, normal and emergency medical facilities, hospital, clinic etc.	1 Bed room, dressing room, toilet, hospital ward and cabin, cell blocks of jail	2.0	1.8
		2 Office room, staff room.	2.5	2.7
		3 X-ray room, operating room, utility room, reading room without book storage.	2.5	4.5
		4 Class room, lecture room, lounge, cafeteria, restaurant.	3.0	2.7
		5 Laboratory, kitchen, laundry	3.0	4.5
		6 Balcony, corridor, lobby, reading room with book storage, staircase	4.0	4.5
		7 Assembly area, fire escape, store room, projection room.	5.0	4.5
		8 Stack room for book	6.5[3]	7.0
ASSEMBLY (Occupancy - E)	Library, auditorium, cinema, lecture hall, restaurant, bar, mosque, church (E1 – E4)	1 Assembly room: with fixed seat	3.0	2.7
		without fixed seat	5.0	4.5
		2 Stages and projection room	5.0	4.5
		3 Library: Reading room – without book storage	2.5	4.5
		with book storage	4.0	4.5
		Stack room for book	6.5[3]	7.0
	Stadium, art gallery, grand stand, museum, gymnasium (E5)	1 With fixed seats	3.0	2.7
		2 Without fixed seats	5.0	4.5
		3 Corridor, stair and passage way	5.0	4.5

BUILDING			w [1]	P [2]
Occupancy		Use of floor	kN/m²	kN
BUSSINESS AND MERCANTILE (Occupancy - F)	Office, bank, laboratory etc.	1 General office room, banking hall	3.0	9.0[5]
		2 Laboratory, kitchen	3.0	4.5
		3 Computer, bussiness machine room	3.5	9.0[5]
		4 File room, filing and storage space	6.0	4.5
		5 Vaults in office and bank	5.0	4.5
		6 Telephone exchange	6.0	4.5
	Shop, market, departmental store	1 Retail store	4.0	3.6
		2 Wholesale store	6.0	13.0[5]
		3 Storage: light	6.0	4.5
		heavy	12.0	14.0[5]
INDUSTRIAL, STORAGE & HAZARDOUS (Occupancy - G, H & J)	Workshop, factory, warehouse	1 Light workroom without storage	3.0	2.7
		2 Machinery hall & circulation area	4.0	4.5
		3 Factory, workshop etc.	5.0	4.5
		4 Manufacturing: light	6.0	4.5
		heavy	12.0	9.0[5]
		ice	15.0	9.0[5]
		5 Printing plant: Press room	7.0	11.0[5]
		Composing and linotype room	5.0	9.0[5]
		Paper storage room	12.0	9.0[5]
		6 Motor room, fan room etc. including the weight of machinery	7.5	4.5
		7 Cold storage, grain storage	15.0	9.0[5]
		8 Storage warehouses: light	6.0	4.5[5]
		heavy	12.0	9.0
		9 Foundries	20.0	12.0
MISCELLANEOUS (Occupancy - K)	Garage, car park, vehicle access ramp	1 Repair workshop for all types of vehicles.	5.0	9.0
		2 Driveway, ramp and parking for vehicles with mass > 2500 kg.	5.0	–[4]
		3 Car parking and ramp for passenger car and light vehicles having mass ≤ 2500 kg	2.5	–[4]
All OCCUPANCIES (Occupancy - A, H, J, K)	Special loads and Miscellaneous live loads (if not specified above)	1 Bed room, toilet, dressing room	2.0	1.8
		2 Office room, staff room	2.5	2.7
		3 Kitchen, laundry, lounge, game room, cafeteria, restaurant	3.0	4.5
		4 Balcony, corridor, passage way, retail store, staircase	4.0	4.5
		5 Assembly area, store room, fire escape, projection room	5.0	4.5
		6 Drill room, drill hall	5.0	9.0
		7 Armories, boiler room and machine room including weight of machinery	7.5	4.5
		8 Airport hangars	7.0	12.0[5]

Note: (1) w : Uniformly distributed load in kN/m². This load shall not be applied simultaneously with the concentrated load, P.

(2) P : A single concentrated load, in kN, assumed to act over an area of 300 mm x 300 mm unless otherwise specified in Note (5) below. Except as indicated by Note (5), these concentrated loads need not be considered for the floors capable of laterally distributing the load, e.g. reinforced concrete slabs

(3) Use a distributed load of 2.4 kN/m² for each metre of stack height but not less than 6.5 kN/m².

(4) See Sec 2.3.3.2 for values, numbers, and spacing of these concentrated loads.

(5) These loads shall be applied over an area of 750 mm x 750 mm on all types of floors including reinforced concrete slab.

Table A.03 (a): Design loads for various occupancy (According to IBC).[A.3]

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
1. Apartments (see residential)	—	—
2. Access floor systems Office use Computer use	 50 100	 2,000 2,000
3. Armories and drill rooms	150 [m]	—
4. Assembly areas Fixed seats (fastened to floor) Follow spot, projections and control rooms Lobbies Movable seats Stage floors Platforms (assembly) Other assembly areas	 60 [m] 50 100 [m] 100 [m] 150 [m] 100 [m] 100 [m]	—
5. Balconies and decks[h]	Same as occupancy served	—
6. Catwalks	40	300
7. Cornices	60	—
8. Corridors First floor Other floors	 100 Same as occupancy served except as indicated	—
9. Dining rooms and restaurants	100 [m]	—
10. Dwellings (see residential)	—	—
11. Elevator machine room grating (on area of 2 inches by 2 inches)	—	300
12. Finish light floor plate construction (on area of 1 inch by 1 inch)	—	200
13. Fire escapes On single-family dwellings only	100 40	—
14. Garages (passenger vehicles only) Trucks and buses	40 [m] See Section 1607.7	Note a
15. Handrails, guards and grab bars	See Section 1607.8	
16. Helipads	See Section 1607.6	
17. Hospitals Corridors above first floor Operating rooms, laboratories Patient rooms	 80 60 40	 1,000 1,000 1,000
18. Hotels (see residential)	—	—
19. Libraries Corridors above first floor Reading rooms Stack rooms	 80 60 150[b, m]	 1,000 1,000 1,000
20. Manufacturing Heavy Light	 250 [m] 125 [m]	 3,000 2,000
21. Marquees	75	—
22. Office buildings Corridors above first floor File and computer rooms shall be designed for heavier loads based on anticipated occupancy Lobbies and first-floor corridors Offices	 80 — 100 50	 2,000 — 2,000 2,000

(continued)

Table A.03 (b): Design loads for various occupancy (According to IBC).

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
23. Penal institutions		
Cell blocks	40	—
Corridors	100	
24. Recreational uses:		
Bowling alleys, poolrooms and similar uses	75[m]	
Dance halls and ballrooms	100[m]	
Gymnasiums	100[m]	—
Reviewing stands, grandstands and bleachers	100[c, m]	
Stadiums and arenas with fixed seats (fastened to floor)	60[c, m]	
25. Residential		
One- and two-family dwellings		
Uninhabitable attics without storage[i]	10	
Uninhabitable attics with storage[i, j, k]	20	
Habitable attics and sleeping areas[k]	30	
All other areas	40	—
Hotels and multifamily dwellings		
Private rooms and corridors serving them	40	
Public rooms[m] and corridors serving them	100	
26. Roofs		
All roof surfaces subject to maintenance workers		300
Awnings and canopies:		
Fabric construction supported by a skeleton structure	5 nonreducible	
All other construction	20	
Ordinary flat, pitched, and curved roofs (that are not occupiable)	20	
Where primary roof members are exposed to a work floor, at single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs:		
Over manufacturing, storage warehouses, and repair garages		2,000
All other primary roof members		300
Occupiable roofs:		
Roof gardens	100	
Assembly areas	100[m]	
All other similar areas	Note 1	Note 1
27. Schools		
Classrooms	40	1,000
Corridors above first floor	80	1,000
First-floor corridors	100	1,000
28. Scuttles, skylight ribs and accessible ceilings	—	200
29. Sidewalks, vehicular drive ways and yards, subject to trucking	250[d, m]	8,000[e]

(continued)

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
30. Stairs and exits		
One- and two-family dwellings	40	300[f]
All other	100	300[f]
31. Storage warehouses (shall be designed for heavier loads if required for anticipated storage)		—
Heavy	250[m]	
Light	125[m]	
32. Stores		
Retail		
First floor	100	1,000
Upper floors	75	1,000
Wholesale, all floors	125[m]	1,000
33. Vehicle barriers	See Section 1607.8.3	
34. Walkways and elevated platforms (other than exitways)	60	—
35. Yards and terraces, pedestrians	100[m]	—

For SI: 1 inch = 25.4 mm, 1 square inch = 645.16 mm^2,
1 square foot = 0.0929 m^2,
1 pound per square foot = 0.0479 kN/m^2, 1 pound = 0.004448 kN,
1 pound per cubic foot = 16 kg/m^3.

a. Floors in garages or portions of buildings used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of Table 1607.1 or the following concentrated loads: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, 3,000 pounds acting on an area of 4.5 inches by 4.5 inches; (2) for mechanical parking structures without slab or deck that are used for storing passenger vehicles only, 2,250 pounds per wheel.

b. The loading applies to stack room floors that support nonmobile, double-faced library book stacks, subject to the following limitations:
1. The nominal bookstack unit height shall not exceed 90 inches;
2. The nominal shelf depth shall not exceed 12 inches for each face; and
3. Parallel rows of double-faced book stacks shall be separated by aisles not less than 36 inches wide.

c. Design in accordance with ICC 300.

d. Other uniform loads in accordance with an approved method containing provisions for truck loadings shall also be considered where appropriate.

e. The concentrated wheel load shall be applied on an area of 4.5 inches by 4.5 inches.

f. The minimum concentrated load on stair treads shall be applied on an area of 2 inches by 2 inches. This load need not be assumed to act concurrently with the uniform load.

g. Where snow loads occur that are in excess of the design conditions, the structure shall be designed to support the loads due to the increased loads caused by drift buildup or a greater snow design determined by the building official (see Section 1608).

h. See Section 1604.8.3 for decks attached to exterior walls.

i. Uninhabitable attics without storage are those where the maximum clear height between the joists and rafters is less than 42 inches, or where there are not two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses. This live load need not be assumed to act concurrently with any other live load requirements.

(continued)

Table A.04: Design loads for various occupancy (According to UBC).[A.4]

USE OR OCCUPANCY		UNIFORM LOAD[1] (psf)	CONCENTRATED LOAD (pounds)
Category	Description	× 0.0479 for kN/m²	× 0.004 48 for kN
1. Access floor systems	Office use	50	2,000[2]
	Computer use	100	2,000[2]
2. Armories		150	0
3. Assembly areas[3] and auditoriums and balconies therewith	Fixed seating areas	50	0
	Movable seating and other areas	100	0
	Stage areas and enclosed platforms	125	0
4. Cornices and marquees		60[4]	0
5. Exit facilities[5]		100	0[6]
6. Garages	General storage and/or repair	100	[7]
	Private or pleasure-type motor vehicle storage	50	[7]
7. Hospitals	Wards and rooms	40	1,000[2]
8. Libraries	Reading rooms	60	1,000[2]
	Stack rooms	125	1,500[2]
9. Manufacturing	Light	75	2,000[2]
	Heavy	125	3,000[2]
10. Offices		50	2,000[2]
11. Printing plants	Press rooms	150	2,500[2]
	Composing and linotype rooms	100	2,000[2]
12. Residential[8]	Basic floor area	40	0[6]
	Exterior balconies	60[4]	0
	Decks	40[4]	0
	Storage	40	0
13. Restrooms[9]			
14. Reviewing stands, grandstands, bleachers, and folding and telescoping seating		100	0
15. Roof decks	Same as area served or for the type of occupancy accommodated		
16. Schools	Classrooms	40	1,000[2]
17. Sidewalks and driveways	Public access	250	[7]
18. Storage	Light	125	
	Heavy	250	
19. Stores		100	3,000[2]
20. Pedestrian bridges and walkways		100	

[1]See Section 1607 for live load reductions.
[2]See Section 1607.3.3, first paragraph, for area of load application.
[3]Assembly areas include such occupancies as dance halls, drill rooms, gymnasiums, playgrounds, plazas, terraces and similar occupancies that are generally accessible to the public.
[4]When snow loads occur that are in excess of the design conditions, the structure shall be designed to support the loads due to the increased loads caused by drift buildup or a greater snow design as determined by the building official. See Section 1614. For special-purpose roofs, see Section 1607.4.4.
[5]Exit facilities shall include such uses as corridors serving an occupant load of 10 or more persons, exterior exit balconies, stairways, fire escapes and similar uses.
[6]Individual stair treads shall be designed to support a 300-pound (1.33 kN) concentrated load placed in a position that would cause maximum stress. Stair stringers may be designed for the uniform load set forth in the table.
[7]See Section 1607.3.3, second paragraph, for concentrated loads. See Table 16-B for vehicle barriers.
[8]Residential occupancies include private dwellings, apartments and hotel guest rooms.
[9]Restroom loads shall not be less than the load for the occupancy with which they are associated, but need not exceed 50 pounds per square foot (2.4 kN/m²).

Table A.05 (a): Design loads for various occupancy (According to Euro Code).[A.5]

Category	Specific use	Example
A	Areas for domestic and residential activities	Rooms in residential buildings and houses; bedrooms and wards in hospitals; bedrooms in hotels and hostels kitchens and toilets.
B	Office areas	
C	Areas where people may congregate (with the exception of areas defined under category A, B and D[1)])	**C1**: Areas with tables, etc e.g. areas in schools, cafes, restaurants, dining halls, reading rooms, receptions **C2**: Areas with fixed seats, e.g. areas in churches, theatres or cinemas, conference rooms, lecture halls, assembly halls, waiting rooms, railway waiting rooms. **C3**: Areas without obstacles for moving people, e.g. areas in museums, exhibition rooms, etc. and access areas in public and administration buildings, hotels, hospitals, railway station forecourts **C4**:Areas with possible physical activities, e.g. dance halls, gymnastic rooms, stages . **C5**:Areas susceptible to large crowds, e.g. in buildings for public events like concert halls, sports halls including stands, terraces and access areas and railway platforms.
D	Shopping areas	**D1**: Areas in general retail shops **D2**: Areas in department stores.
[1)] Attention is drawn to 6.3.1.1(2), in particular for C4 and C5. See EN 1990 when dynamic effects need to be considered. For Category E, see Table 6.3		
NOTE 1. Depending on their anticipated uses, areas likely to be categorised as C2, C3, C4 may be categorised as C5 by decision of the client and/or National annex.		

Table A.05 (b): Design loads for various occupancy (According to Euro Code).

Categories of loaded areas	**q_k [kN/m^2]**	**Q_k [kN]**
Category A		
- Floors	1,5 to 2,0	2,0 to 3,0
- Stairs	2,0 to 4,0	2,0 to 4,0
- Balconies	2,5 to 4,0	2,0 to 3,0
Category B	2,0 to 3,0	1, 5 to 4,5
Category C		
- C1	2,0 to 3,0	3,0 to 4,0
- C2	3,0 to 4,0	2,5 to 7,0 (4,0)
- C3	3,0 to 5,0	4,0 to 7,0
- C4	4,5 to 5,0	3,5 to 7,0
- C5	5,0 to 7,5	3,5 to 4,5
Category D		
-D1	4,0 to 5,0	3,5 to 7,0 (4,0)
-D2	4,0 to 5,0	3,5 to 7,0

NOTE: Where a range is given in this table, the value may be set by the National annex. The recommended values, intended for separate application, are underlined. q_k is intended for the determination of general effects and Q_k for local effects. The National annex may define different conditions of use of this Table.

REFERENCES

2.1 *Minimum Thicknesses for Beams and One-Way Slabs*, Figure 3-1, Simplified Design of Reinforced Concrete Buildings, Mahmoud E. Kamara and Lawrence C. Novak, Fourth Edition.

2.2 *Minimum Slab Thickness for Two-Way Slab Systems*, Figure 4- 4 , Simplified Design of Reinforced Concrete Buildings, Mahmoud E. Kamara and Lawrence C. Novak, Fourth Edition.

3.1 *Coefficients for Negative Moments in Slabs*, Method-3, Table-1, Building Code Requirements for Reinforced Concrete, American Concrete Institute (ACI 318-63).

3.2 *Coefficients for Dead Load Positive Moments in Slabs*, Method-3, Table-2, Building Code Requirements for Reinforced Concrete, American Concrete Institute (ACI 318-63).

3.3 *Coefficients for Live Load Positive Moments in Slabs*, Method-3, Table-3, Building Code Requirements for Reinforced Concrete, American Concrete Institute (ACI 318-63).

A.1 *Minimum Uniformly Distributed Live Loads, Lo, and Minimum Concentrated Live Loads*, Table 4-1, Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers, (ASCE or SEI 7-10).

A.2 *Live Loads for Various Occupancies*, Table 6.2.3, Bangladesh National Building Code, (BNBC 2006).

A.3 *Minimum Uniformly Distributed Live Loads, Lo, and Minimum Concentrated Live Loads*, Table 1607.1, International Building Code (ICC or IBC 2012).

A.4 *Uniform and Concentrated Loads*, Table 16-A, Uniform Building Code, (UBC 1997 Volume 02).

A.5 *Imposed loads on floors, balconies and stairs in buildings,* Table 6.1-6.2, Design of Concrete Structures, Euro/British Code, (EN 1991).

i) Design of Concrete Structures, George Winter, C. E. O'Rourke, L. C. Urquhart and Arthur H. Nilson, Seventh Edition.

ii) Design of Concrete Structures, Arthur H. Nilson, David Darwin and Charles W. Dolan, Thirteenth Edition.

iii) Simplified Design of Reinforced Concrete Buildings, Mahmoud E. Kamara and Lawrence C. Novak, Fourth Edition.

iv) Building Code Requirements for Reinforced Concrete, American Concrete Institute (ACI 318-11).

v) Building Code Requirements for Reinforced Concrete, American Concrete Institute (ACI 318-63).