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CHAPTER 1

INTRODUCTION

1.1 General

Reinforced concrete structures have considerable compressive strength as compared to most other materials. In addition to the high compressive strength, reinforced concrete structures are durable, versatile, and have comparatively less maintenance cost when compared to steel structures. They also provide good resistance against fire and water damage, and have excellent potential for long service life (Wight, 2008)

In the design and construction of reinforced concrete structures material cost is an important issue. The main factors affecting cost are the amount of concrete and steel reinforcement required. It is, therefore, suitable to design reinforced concrete structures lighter, while still satisfying the serviceability and strength requirements. Besides the material cost, labor and formwork costs are also substantial.

Good engineers are those having the ability of designing the economical structures without compromising its function or despoiling structural constraints. The conventional approach to design reinforced concrete members does not fully optimize the use of materials.

Most of the structural designs are based on the past experience of the engineer, who selects geometry of the section and material grades by comparing past experience. This gives rise to fixed guidelines for preliminary designs (Zaforteza, 2009).This process is normally of high cost in terms of time, human exertion and material usage, which makes structural optimization procedures using artificial intelligence a clear substitute to the designs based on experience.(Coello, 1997)

Optimization of reinforced concrete members is a complex problem, because it involves the large number of variables in the design process, the different values of

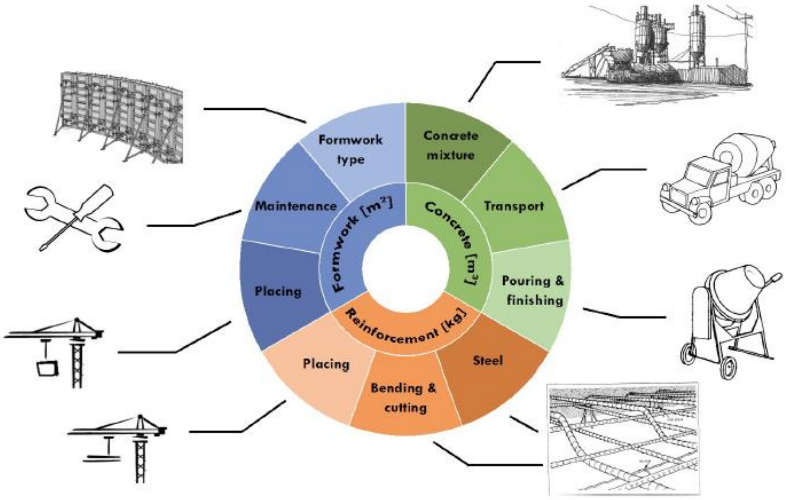


Figure 1: Factors Affecting Cost

these variables and the various reinforcement details available for a single design problem. (Wight, 2008)

1.2 Problem statement

In general Structural designer have to consider these four types of design variables; Material design variables such as the type of concrete and grade of steel, topological variables such as number of members in a structure, geometric layout variables such as the length of the member and cross-sectional variables such as dimensions of section.

Obtaining an optimal solution within a large space of possible solutions is very complex to solve by hand, and even traditional approaches fail in obtaining such solution. This is due to the large number of design variables, their interaction with each other and their influence on the final cost.

Typically, the design is limited by some constraints such as the selection of material, strength to be required, displacements, applied loads, support conditions and achieving requirements as stated in codes of practice.

The optimization of reinforced concrete (RC) members is very complicated due to the absence of standard RC sections like those in its steel counterpart. Furthermore, RC sections deal with both discrete and continuous variables. Moreover, a large number of possible section designs can still achieve the strength and serviceability required. The large number of design possibilities adds more complications to the problem at hand. We will consider an approach of SQP (Sequential quadratic programming) for optimizing the cross section and reinforcement of reinforced concrete frames.

In an optimization procedure, the definition of the cost function may be considered the most important decision, which represents the aim of the problem. Therefore, it is essential to define a cost function that represents the most influential cost components and more importantly, is applicable to the variety of optimizing problem.

In concrete structures, at least three different cost items should be considered in an optimization problem: cost of concrete, cost of steel and cost of formwork.

1.3 Motivation

Design optimization methods have been used to obtain more economical designs since 1970s (Pics, 1970) - (Glover, 1975). Numerous algorithms have been developed for accomplishing the optimization problems in the last five decades. The early works on the topic mostly use mathematical programming techniques or optimality criteria with continuous design variables. These methods utilize gradient of functions to search the design space. Today's competitive world has forced the engineers to realize more economical designs and designers to develop more effective optimization techniques.

1.4 Objective

The main objective is to develop an optimization model that is capable of obtaining the optimum design for reinforced concrete frames in terms of layout, cross section dimensions and reinforcement details. The optimization is carried out using Ant

Colony Algorithm and SQP Algorithm, while still satisfying the strength and serviceability constraints of the American Concrete Institute Building Code Requirements for Structural Concrete and Commentary (ACI318-14).

The objectives of this study are:

- Develop a computer program which gives economic design of regular RCC building satisfying the ACI strength and serviceability constraints.
- Carry out validation and verification of the developed model.
- Compare the optimized design with typical design results.
- Draw conclusions and recommendations.

1.5 Scope and Limitations

The scope and limitations of this study are as following:

1.5.1 **Regularity of Structure**

The desired model would be only applicable to regular RCC single story buildings. There should be no discontinuity in the structure. Moreover, only rectangular panels are allowed avoiding the offset of columns.

1.5.2 **Serviceability and Strength requirement**

Design conforms to the strength and serviceability constraints of the (ACI318-14).

1.5.3 **Floor System**

Floor system consists of the beams running continuously both in longitudinal and transverse direction. Slab-beam floor system is only considered in our design.

1.5.4 **Torsion**

Beams are not designed for the torsion.

1.5.5 Shear Design

Optimization is limited to flexural and axial reinforcement. It is assumed that the design for shear loads does not alter the optimal design decision variables. (Andres Guerra, 2006)

1.5.6 Elastic/ Linear Behavior

Linear behavior of RCC frames is considered.

1.5.7 Slender Columns

Since, L/h is less than 12 so, columns are not slender. Therefore, slenderness ratio is not considered in our study.

1.5.8 Joint Detailing

It is also assumed that the optimal solution is not sensitive to connection detailing. For structures in Seismic (Design Category A, B, and C as classified in the ASCE Standard (SEI/ASCE 7-98) this assumption is acceptable.

1.5.9 Structural Elements considered

Optimization process includes optimizing both cross sectional dimensions and steel ratios for beams, columns and slab in the frame. Foundations, joints, staircase etc. are considered out of scope of this study.

1.5.10 Loading Conditions

Since the ratio of factored live load to factored dead load is kept less than 3/4 (0.75). Therefore, Pattern Loading is not applicable and is not considered. Assumed all the loading is uniformly distributed on the entire structure.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction to Optimization

2.1.1 Concept of Optimization

Optimization is the process of getting the best acceptable alternative from a set of possible available alternatives (Stuzzle, 2010).

Therefore it is a system, shown in Figure (2),that relies on available alternatives and constraints as input, then processes these inputs making use of an optimization technique and results in the optimum solution as an output.

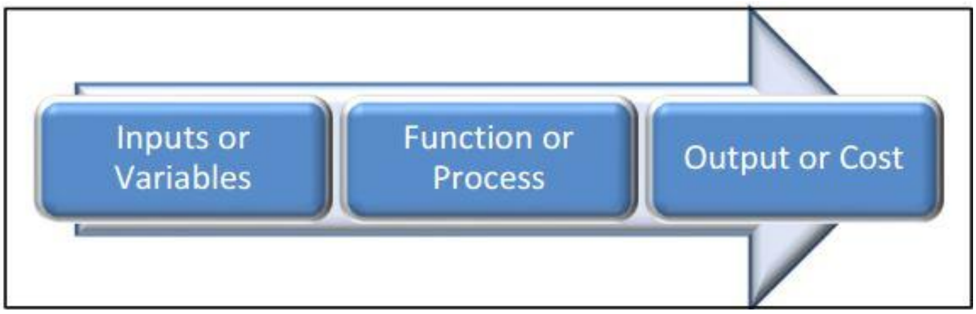


Figure 2: Optimization Process as a system

2.1.2 Global Optimality versus Local Optimality

This has some special importance in reinforced concrete frame optimization problems since the number of possible variable combinations for the simplest of reinforced concrete frame is practically infinite. These complications gives several local optimal solutions having one of them being the best, i.e. the global optimum.

Figure (3) further exemplifies this concept. For the simplest mathematical optimization problem of continuous functions, one can easily find more than one local minima (the white stars) in domain of the function. Out of these one is considered as the global optimum (the black star), which the algorithm seeks to find.(Valley, 2009).

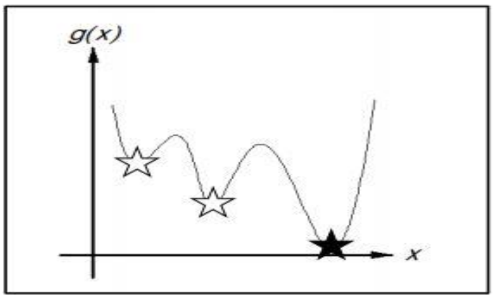


Figure 3: Difference between Global optimum and Local optimum

2.1.3 Categories of Optimization

Optimization techniques can be categorized into seven categories, shown in Figure (3). For example, a dynamic optimization problem can be either constrained or unconstrained. In the addition, few variables may be discrete and others continuous.

The different types of optimization algorithms are discussed below. Trial-and-error optimization talks about the process of adjusting variables that affect the output without having much knowledge about the process that produces the output. In contrast, a mathematical formula explains the objective function in function optimization. Various mathematical manipulations of the function point to the optimal solution.If there is only one variable, the optimization is one-dimensional. A problem comprising of more than one variable requires multidimensional optimization. Optimization becomes increasingly tougher as the number of dimensions increases. Many multidimensional optimization approaches simplify to a series of one-dimensional approaches.

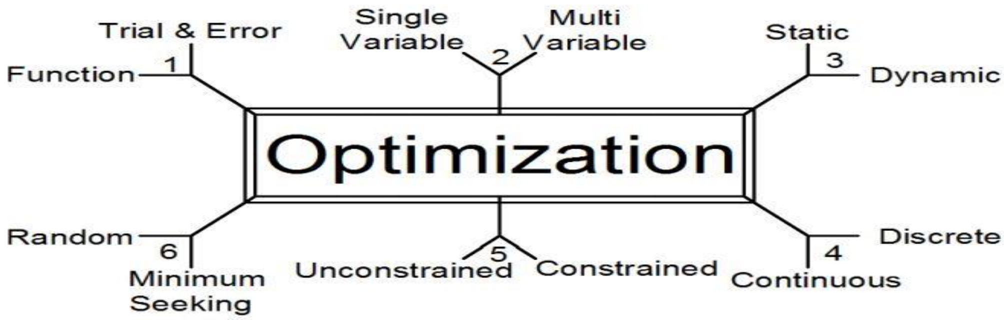


Figure 4: Categories of Optimization Techniques

Dynamic optimization means that the result is a function of time, while static means that the output is not time dependent. For instance: Finding the fastest route is a dynamic problem whose solution depends on the time of day, the weather, accidents, and so on. The static problem is problematic to solve for the best solution, but the added dimension of time increases the challenge in solving the dynamic problem.

Optimization can also be differentiated by either discrete or continuous variables. Discrete variables contain only a finite number of possible values, whereas continuous variables comprises of an infinite number of possible values. If we are deciding in what order to attack a series of tasks on a list, discrete optimization is used. Discrete variable optimization is also known as combinatorial optimization, because the optimum solution contains a certain combination of variables from the finite pool of all possible variables. However, if we are searching for the minimum value of a function on a number line, it is more suitable to view the problem as continuous.

Sometimes variables have limits or constraints. Constrained optimization incorporates variable equalities and inequalities into the cost function. Unconstrained optimization allows the variables to choose any value. A constrained variable often converts into an unconstrained variable through the conversion of variables. Most numerical optimization routines work better with unconstrained variables.

Some algorithms try to lessen the cost by starting from an initial set of variable values. These minimum seekers easily get stuck in local minima but tend to be fast. They are the conventional optimization algorithms and mostly based on calculus methods. Moving from one variable set to another is based on some determinant order of steps. On the other hand, random methods use some probabilistic calculations to find variable sets. They tend to be slower but having chance of greater success in finding the global minimum.

2.1.3.1 Classification based on the physical structure of the problem

On the basis of physical structure of the problem we can classify the optimization problems into optimal control and non-optimal control problems.

2.1.3.2 Classification based on the nature of the equations involved

On the basis of nature of expressions for the objective function and the constraints, we can classify the optimization problems into linear, nonlinear, geometric and quadratic programming problems.

2.1.3.3 Classification based on the permissible values of the decision variables

On the basis of permissible values of the decision variable, we can classify the optimization problems as integer and real-valued programming problems.

2.1.3.4 Classification based on deterministic nature of the variables

On the basis of deterministic nature of variable, we can classify the optimization problems as deterministic and stochastic programming problems.

2.1.3.5 Classification based on separability of the functions

On the basis of separability of the objective function and constraint functions, we can classify optimization problems as separable and non-separable programming problems.

2.1.3.6 Classification based on the number of objective functions

On the basis of this classification of objective functions, we can classify as single and multi objective programming problems.

2.1.4 **Heuristic Optimization Techniques**

1. The problems are solved iteratively.
2. They are capable of optimizing systems which have continuous, discrete or integer design variables.
3. The solution is not always the global optimum that totally depends upon the nature of the problem.
2. The problem does not get trapped in local optimums.
3. They do not necessarily produce the same solution each time.

2.2 Common Heuristic Optimization algorithms

Most of the algorithms are based on numerical linear and nonlinear programming methods that require substantial gradient information and usually seek to improve the solution in the neighborhood of a starting point. These numerical optimization algorithms provide a useful strategy to obtain the global optimum in simple and ideal models. Many real-world engineering optimization problems, however, are very complex in nature and quite difficult to solve using these algorithms. If there is more than one local optimum in the problem, the result may depend on the selection of an initial point, and the obtained optimal solution may not necessarily be the global optimum. Furthermore, the gradient search may become difficult and unstable when the objective function and constraints have multiple or sharp peaks. The computational drawbacks of existing numerical methods have forced researchers to rely on meta-heuristic algorithms based on simulations to solve engineering optimization problems. The common factor in meta-heuristic algorithms is that they combine rules and randomness to imitate natural phenomena. To solve difficult and complicated real-world optimization problems, however, new heuristic and more powerful algorithms based on analogies with natural or artificial phenomena must be explored. The following sections, will give a general idea of some existing meta-heuristic algorithms.

2.2.1 Genetic Algorithm (GA)

Genetic Algorithms technique can be used for both constrained and unconstrained optimization problems. It generates a population of points after each iteration and then leads to the best optimal solution.GAs do not need derivatives of functions rather it deal with discrete optimum design problems. However, GA doesn't work well when function is complex and it chooses only better solution while comparing with other solution. Sometimes GAs leads us to the local optima or some random points rather than to the global optima. It is not much efficient in terms of speed of convergence for some specific optimization problems.

2.2.2 Simulating Annealing Algorithm (SA)

For approximating the global optimum of a given function a probabilistic technique which is used is simulating annealing (SA). It is used when search space is very large. This technique is suitable where it is more important to find a nearest solution than the precise global optimum solution. The SA algorithm is developed on the basis of analogy between the annealing of solids and finding the solutions to optimization problems. The method was developed by Scott. Kirkptrick and Mario P. Vechi(1983).

2.2.3 Ant Colony Optimization Algorithm (ACO)

To solve discrete optimum structural problems an application of ant behavior to the computational algorithms is used that is called Ant colony optimization (ACO). It works very well in graphs with changing topologies. Some extra artificial characteristics like memory, visibility and discrete time are also available in this type of algorithm. ACO was originally developed by Dorigo(1992) for optimization problems.

2.2.4 Harmony Search Optimization Algorithm (HS)

Zong Woo Geem and Lee developed a harmony search (HS) meta-heuristic algorithm that was conceptualized using the musical process of searching for a perfect state of harmony. The harmony in music is analogous to the optimization solution vector, and the musician's improvisations are analogous to local and global search schemes in optimization techniques. It does not require initial value setting for the variables and it is free from divergence.

2.3 Structural Optimization

Optimal structural design is becoming increasingly important due to restricted material resources, and its impact on environment and technological competition, all these demand light weight, high performance and most importantly low life-cycle-cost structures. The main concerns of structural engineers are the design of a safe and economical structure. Economy in design can be obtained through an optimization procedure with the aim of to find the most efficient structure which will satisfy the chosen criteria. Combining an optimization procedure with structural modeling, and analysis and design methods, and then augmenting them with the cost of systems and materials in an exclusive process will lead to the development of a powerful optimization system.

Modern structural optimization has its roots in the 1960s with Lucien Schmidt‘s seminal paper. While the 1960s and 1970s were characterized by difficulties in solving even small optimization problems (forgetting for the moment the optimal criteria methods), the 1990s were defined by discussions regarding the use of mathematical programming methods for solving large systems.

From the 1960s a considerable amount of research has been published in the area of structural optimization, with the majority of these papers dealing with reducing the weight of a structure. While the weight of a structure comprises a considerable part of the cost, a minimum weight design is not necessarily the minimum cost of a design. Only a small part of the papers published on structural optimization cope with the cost optimization problem, most of them cope with structural elements such as beams, even though some journal papers have been published on the cost optimization of realistic 3D structures. As such, it is necessary to do research on the cost Optimization of realistic 3D structures, especially massive structures with hundreds of members where optimization can result in considerable savings, the result of such research efforts will be of great importance to structural engineers.

In steel structures optimization is the problem of reducing weight, the optimization of reinforced concrete structures must be expressed as a cost minimization problem due to involvement of different materials. Only a small portion of the hundreds of papers published on the optimization of steel structures cope with optimizing costs; while reducing the weight does not necessarily lead to the minimum cost and in actual, a minimum weight design might not be a minimum cost design. Aside from the cost of materials, many other factors affect the total construction cost of a structure.

Present days, research into structural optimization has focused on changing the geometry (shape) and topology of the structural conformation because geometrical changes require a redefinition of the finite element mesh. Topological changes, which comprises of adding or eliminating parts as well as creating holes, pose even more cumbersome challenges in converting the structural design into a manageable optimization problem.

2.4 Background of the Study

Optimization is generally finding out the best results for a given problem under some specific circumstances. Engineers have to take many technological and managerial decisions at many stages in the design, construction and maintenance of any engineering system with the main objective being either to maximize the desired benefit or reduce the effort required.

A structure in mechanics can be defined as accumulation of materials, which is planned to bear the loads. Optimization is generally sorting out the solution to get the best. Thus, structural optimization can be defined as making an accumulation of materials which is able to bear the loads in the best way. Structural optimization problems can be illusorily simple to formulate, and can be written as:

$$\text{Min } f(x) \text{ subjected to } g(x) \leq 0$$

in this equation x represents the set of the variables, $g(x)$ is the set of constraints and $f(x)$ is the objective function. Structural optimization can be categorized into geometry, topology, and sizing optimization. Sizing (cross-sectional) optimization is to find out the optimal cross sectional properties of members in a frame structure, or the optimal thickness of the slab. It has the goal of maximizing the performance of a structure in terms of its weight and overall stiffness or strength, while fulfilling its equilibrium condition and the design constraints. The cross sectional parameters of the members of the structure are the design variables. In sizing optimization, the design domain is fixed during this process, whereas in shape optimization, the goal is to find the optimal shape of the design domain, which increases its performance. The geometry of the design domain is not fixed; it is a design variable, which means that in shape optimization, topology of the domain is fixed and only the boundaries of the design domain are variable. The topology optimization of continuum structures means finding the locations and optimal number of the components within the continuum design domain. In topology optimization, topology and shape of a structure both are the design variables. In the Topology optimization problems, the layout properties of the structures such as the bay width of a frame, are usually called layout optimization in the literature, are the design variables.

2.5 Optimization of Reinforced Concrete

In Structural Design mostly the area of interest is to find out the general geometric layout of the system that supports the anticipated design actions. By optimizing the overall layout of the structure such a design can be achieved. The most problematic in structural optimization is Layout optimization. It is also very important, due to the fact that it gives much higher material saving as compare to the optimization of the cross-sections of the elements of the structure. Selecting a proper geometric layout has much importance in a comprehensive structural optimization process, as it affects all the subsequent stages of the design procedure.

Span lengths as geometric layout variables, are determined based on the architectural requirements and constraints, in preliminary geometric design of the buildings. In this case, choosing the best possible layouts among them can result in a considerable cost saving, as the primary design layout will influence the whole design process. That’s why an optimization procedure in this design phase that takes the related cost elements into account, along with the cost optimization in the detailed design phase, can lead to a comprehensive optimal design procedure.(Sharafi, 2013)

Reinforced concrete (RC) structures’ design optimization is difficult because of the complexity linked with reinforcement design. Also three different cost components steel, concrete and formwork have to be considered in the case of concrete structures, and any little changes in the quantity of any of these items will affect the overall cost of the structure to a great value. Therefore, the problem is basically the choosing of a combination of quantity of reinforcement and suitable values of design variables to get the total cost component minimum(Kaveh & Sabzi, 2011).

Material and construction cost of reinforced concrete can be reduced by using a genetic algorithm design procedure while fulfilling the specifications and limitations of the ACI Code. Beam elements in frame are assessed on the basis of their flexural response by keeping in mind the moment magnification factors caused by frame stability. To assess the feasibility of columns with moment magnification caused by slenderness effects, a rectilinear column strength interaction diagram is used. The specifications and limitations of the ACI Code are devised as a string of constraints to the cost optimization problem and penalizes on the fitness function of the genetic algorithm. (Camp, Pezeshk, & Hansson, 2003)

(Kaveh & Sabzi, 2011)researched that there are two methods to find out the optimum design of reinforced concrete frames: The heuristic big bang-big crunch (HBB-BC), which is based on big bang-big crunch (BB-BC) and a harmony search (HS) scheme to deal with the variable constraint, and The (HPSACO) algorithm, which is a combination of particle swarm with passive congregation, ant colony optimization, and harmony search scheme algorithms. They studied three frames and obtained optimum designs of columns and beams without considering joint detailing or shear reinforcement. The design variables used were simply the cross sectional dimensions of columns, column reinforcement, beam cross sectional dimensions as well as the number and diameter of steel bars used as top and bottom reinforcement not including cut off bars.

(Akin & Saka, 2011)researched the harmony search algorithm to find the optimum detailed design of reinforced concrete continuous beam. He chooses different design variables as the cross section dimensions of beam in each span, the number of longitudinal reinforcement bars and the diameter along the span and supports and also the diameter of shear reinforcement as well as the diameter and number of curtail bars. The values of these variables are obtained from a design pool having discrete values for these variables. The design constraints are followed from ACI 318-05.

(Zaforteza & Vidosa, 2009) used simulated annealing (SA) to study the CO_2 optimization of reinforced concrete frames. In order to minimize pollution, they did a comparison of the optimum design of a reinforced concrete frame to the amount of CO_2 gas emitted. The depth and width of the beams and columns, the type of concrete and grade of steel, as well as the reinforcement of the frame are the defined design variables. For reinforcement detailing, they took shear reinforcement and cut off bars into considerations whereas joint detailing was not.

The Optimum design of reinforced concrete frames on the basis of predetermined section database was studied by(Kwak & Kim, 2008). The study formulates a database of all possible cross sections and sorts them according to their strength. Design variables in a RC section such as the width and depth of the cross section and steel quantity are joined by a single design variable that take away almost all of the limitations of mathematical programming procedures associated with the complex structures.

An ideal technique is based on Genetic algorithm methodology to form practical design considerations like predefined discrete changes in layout of concrete frame members and detailing the placing of reinforcement bars. This genetic modeling method allows the structural engineer to mention allowable combinations of reinforcement bars available sizes. (Rajeev & Krishnamoorthy, 1998)

The *RC-GA* (Genetic Algorithm) design procedure reduces the cost of concrete by minimizing the material while fulfilling the specification and limitations of the ACI code. Beams in the frame were assessed on the basis of their flexural response while keeping in mind the moment magnification factors caused by frame stability. To assess the feasibility of columns with moment magnification due to slenderness effects, a rectilinear column strength interaction diagram had been used. The reduction in structural costs by using the *RC-GA* design method might be not worth mentioning in the total cost of structure, the automatic and systemic confirmation of the ACI Code restrictions can give an amplified level of confidence in integrity of the design. (Camp & Hansson, 2003)

2.6 MATLAB

MATLAB (matrix laboratory)is fourth-generation programming language and a multi-paradigm numerical computing environment. A special programming language developed by Math Works. MATLAB can perform many operations like operation of algorithms, matrix manipulations, developing user interfaces. Plotting of functions and data and interfacing with other programs which are written in other languages, such as Fortran, C++, C, Java and Python.

MATLAB is basically planned for numerical computing. MATLAB is an optional toolbox which utilizes the MuPAD, symbolic engine, which in return gives access to abilities of symbolic computing. To add graphical multi-domain simulation and model-based design for embedded and dynamic systems Simulink is an additional package.

The MATLAB platform is optimized for finding out the solution of engineering and scientific problems. The world's most usual way to express computational mathematics is MATLAB (Matrix based language). MATLAB has built-in graphics which makes it easy to visualize and gain perceptions from data. A vast library of already built toolboxes allows you to get started right away with algorithms necessary to your domain. These MATLAB abilities and features are all thoroughly tested and designed to work together.

2.6.1 **Syntax**

The MATLAB application is developed on the MATLAB scripting language. Mostly, usage of the MATLAB application comprises using the Command Window as a communicating mathematical shell or executing text files which contains MATLAB code.

2.6.1.1 Variables

In MATLAB the variables can be defined by using the assignment operator, `=`. MATLAB is a weak programming language. Since the variables can be assigned without declaring their type therefore, it is a contingent typed language; their type can change when they are treated as symbolic objects. Values come from computation involving values of other variables, come from constants or from the output of a function. For example:

```
>> x = [3*4, pi/2]
x =
   12.0000    1.5708

>> y = 3*sin(x)
y =
   -1.6097    3.0000
```

2.6.1.2 Vectors and Matrices

The colon syntax: is used to define a simple array in MATLAB. **init:increment:terminator**. For instance:

```
>> array = 1:2:9
array =
 1 3 5 7 9
```

It defines a variable named array (or giving a new value to current variable with the name array) which is an array comprising of the values 1, 3, 5, 7, and 9. That is, the array starts at the **init** value (1), increments in each step from the last value by the **increment** value (2), and terminate when it reaches to the **terminator** value (9).

Matrices in MATLAB are defined by sorting out the elements of a row with blank space and using a semicolon to end the each row. The list of elements should be surrounded by square brackets, which we want to enter: []. To approach elements and sub arrays Parentheses () are used.

```
>> A = [16 3 2 13; 5 10 11 8; 9 6 7 12; 4 15 14 1]
A =
 16  3  2 13
  5 10 11  8
  9  6  7 12
  4 15 14  1

>> A(2,3)
ans =
 11
```

2.6.1.3 Structures

Data type of MATLAB is structure data types. A more suitable name is "structure array", because in MATLAB all variables are arrays, where the field name of each and every element is the same. Unfortunately, MATLAB JIT is not able to support MATLAB structures; therefore just a simple bundling of several variables into a structure will come at a cost.

2.6.1.4 Functions

The name of the file should be similar to the name of the first function in the file while developing a MATLAB function. Authentic function names must begin with an alphabetic character, and it can contain numbers, letters or underscores.

2.6.1.5 Function Handles

MATLAB supports elements of lambda calculus by introducing function handles or function references which can be implemented in **.m** files.

2.6.1.6 Classes and Object oriented Programming

As compare to the other languages the syntax and calling conventions are considerably different. MATLAB has reference classes and value classes, depending on either the class has handle as a super-class (for reference classes) or not (for value classes).

Method call behavior is unalike between reference and value classes. For example, the syntax for a call to a method is:

```
object.method();
```

If object is an instance of a reference class then this can alter any member of object .

An example of a simple class is given:

```
classdef hello
    methods
        function greet(this)
            disp('Hello!')
        end
    end
end
```

When that is entered into a file named **hello.m**, then that can be executed with the following commands:

```
>> x = hello;
>> x.greet();
Hello!
```

CHAPTER 3

METHODOLOGY

This chapter explains in detail the procedure we adopted to analyze and design reinforced concrete frames conforming to strength and serviceability constraints of the ACI code. The stepwise procedure used to analyze and design frames can be depicted in the flowchart below.

3.1 Equivalent Frame Method

If two way slabs does not satisfy the limitations of the direct design method, the design moments and reinforcement area must be calculated by the equivalent frame method. In the latter method, the building is divided into equivalent frames in two directions and then analyzed elastically for all conditions of loadings. The difference between the two methods lies in the way in which moments are calculated in the longitudinal and transverse direction. The design by equivalent frame method can be done by following steps.

3.1.1 Description of the equivalent frame

The 3D frame is divided into a series of 2D equivalent frames centered on column. The width of each equivalent frame is limited by the centerlines of the neighboring panels. The complete analysis of 3D frame is done by analyzing the equivalent frames in longitudinal and transverse direction. The equivalent frame contains following parts:

1. The horizontal slab which may include beams
2. The columns extending above and below the slab.
3. Members that transfer moments between horizontal and vertical members.

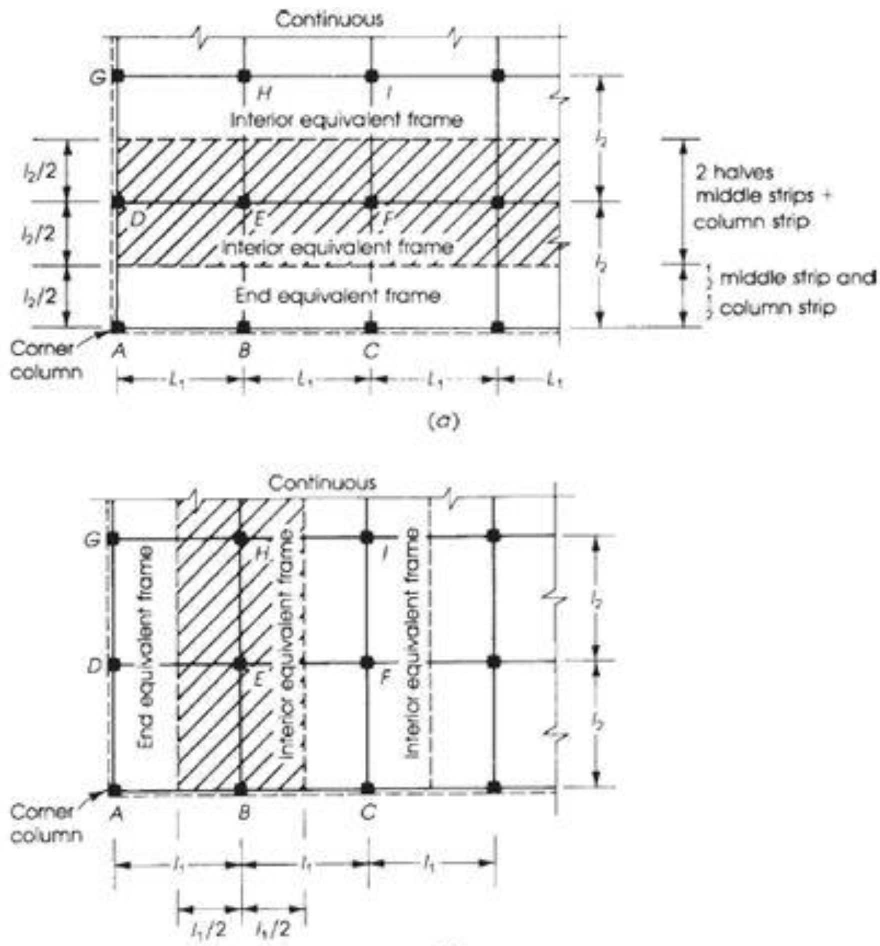


Figure 5: (a) Longitudnal and (b) Transverse Equivalent frames in Plan view

3.1.2 Load assumptions

When of the live load (L.D) to the dead load(D.L) the ratio is less than or equal to 0.75, the structural analysis of the frame can be made with the factored dead and live loads acting on all spans instead of a pattern loading. When the live load (L.D) to the dead load(D.L) ratio is greater than 0.75, pattern loading must be used, considering the following conditions:

1. Only 75% of the full-factored live load may be used for the pattern loading analysis.
2. When two neighboring panels are loaded maximum negative moment is produced at the support between them
3. The maximum positive moment near a mid span is obtained by loading only alternate spans.
4. The design moments should not be less than when full-factored live load is placed on all panels.
5. The critical negative moments are considered to be acting at the face of a rectangular column

3.1.3 Slab-Beam Moment of Inertia

The ACI Code specifies that for analysis of the frame change in the moment of inertia of column and slab-beam should be considered. $\mathbf{K_{sb}}$ represents the combined stiffness of slab and longitudinal beam (if any).The moment of inertia of slab-beams can be assumed equal to the moment of inertia of the slab-beam at the face of the column divided by the term $(\mathbf{1} - \boldsymbol{c}\mathbf{2}/\boldsymbol{\ell}\mathbf{2})\mathbf{2}$, where $\boldsymbol{c}\mathbf{2}$ and $\boldsymbol{\ell}\mathbf{2}$ are measured at right angles to the direction of the span for which moments are being determined.

3.1.4 Column’s Moment of inertia

The ACI Code, Section 13.7.4, states that the moment of inertia of the column is to be assumed infinite from top of the slab to the bottom of the slab beams.

3.1.5 Column Stiffness

Column stiffness is calculated by taking length from the center of slab above to center of slab below. Column moment of inertia is obtained by its cross-section.

The equivalent column consists of the actual columns above and below the slab-beam, plus involved torsional members on each side of the columns ranging to the centerline of the neighboring panels, as shown in Fig.

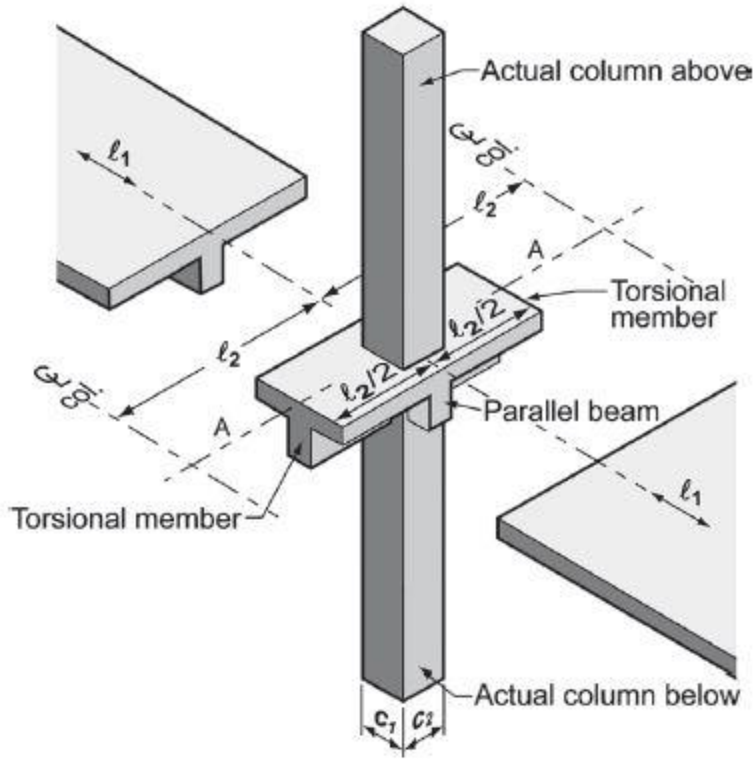


Figure 6: Equivalent Column plus Torsional members

K_{ec} represents the modified column stiffness. The modification depends on lateral members (slab, beams etc) and presence of column in the storey above.

3.1.6 Column Moments

In frame analysis, moments determined for the equivalent columns at the upper end of the column below the slab and at the lower end of the column above the slab must be used in the design of a column.

3.1.7 Negative Moments at the support

The ACI Code, Section 13.7.7, states that for an interior column, the factored negative moment should be taken at the face of column but at a distance not greater than 0.1175l1 from the column's centre. For an exterior column, the factored negative moment is to be taken at a section located at half the distance between the edge of the support and the face of the column.

3.1.8 **Sum of Moments**

A two-way slab floor system that is compatible with direct design method can also be analyzed by the equivalent frame method. To ensure that both methods will produce similar results, the ACI Code, Section 13.7.7, states that the computed moments determined by the equivalent frame method may be reduced in such proportion that the sum of the magnitude of average negative and positive moments used in the design should not cross the total statical moment, Mo.

The effect of 3D frame in to 2D frame is done by use of slab-beam column stiffness K_{sb} and K_{ec} modified column Stiffness

Once a 2D frame is obtained it can be analyzed by any 2D frame analysis.

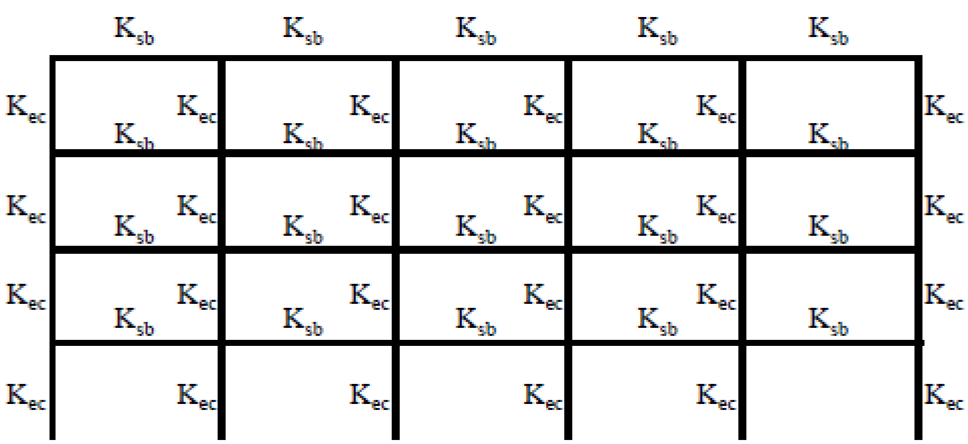


Figure 7: 2D Frame (Equivalent Frame)

3.2 Analyzing the Frame

There are several steps which are followed to analyze and determine the design moments required for 3D frame.

3.2.1 **Minimum Slab Thickness**

ACI code specifies minimum thickness of a slab to limit the deflection. The flexural stiffness of a slab is the main variable on which magnitude of slab deflection depends. The limitations for the deflection can be calculated by using following equations:

α_{fm}= average value of α for beams on the sides of a panel

β = ratio of long to short clear spans

For $0.2 < \alpha_{fm} \leq 2$,

$$h = \frac{l_n\left(0.8 + f_y/200{,}000\right)}{36 + 5\beta(\alpha_{fm} - 0.2)} \quad (f_y \text{ in psi})$$

but not less than 5 in.

For $\alpha_{fm} > 2.0$,

$$h = \frac{l_n\left(0.8 + f_y/200{,}000\right)}{36 + 9\beta} \quad (f_y \text{ in psi})$$

but not less than 3.5 in.

For $\alpha_{fm} \leq 0.2$,

h = minimum slab thickness without interior beams

where

l_n = clear span in long direction measured face to face of columns (or face to face of beams for slabs with beams)

β = ratio of the long to the short clear spans

3.2.2 **Slab-beam moment of inertia**

1. First we will determine the slab-beam moment of inertia $\mathbf{I_{sb}}$ by considering the section given by ACI **8.4.1.8.**

For monolithic or fully composite construction supporting two-way slabs, the portion of slab which is included in beam is on each side of the beam extending at a distance equal to the projection of the beam above or below the slab, whichever is greater, but should not be larger than four times of the slab thickness.

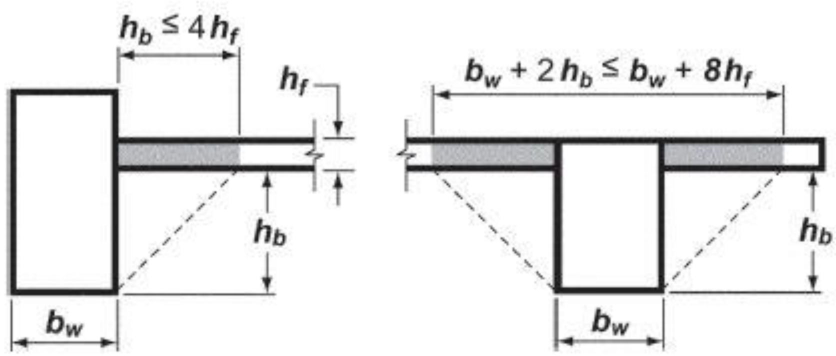


Figure 8: Portion of slab to be included with the beam

2. Then we will determine the k factorby the foolowing formula:

$$k = 5.3 \left(\frac{c_1}{\ell_1} \times \frac{c_2}{\ell_2} \right)^{0.05} \alpha^{0.9} \geq 4.0$$

Where,

$$\alpha = \frac{depth\ at\ drop\ panel}{depth\ of\ slab}$$

C1 = larger of column or capital width at the top in the direction of calculation of moments.

C2 = larger of column or capital width at the top perpendicular to the direction of calculation of moments.

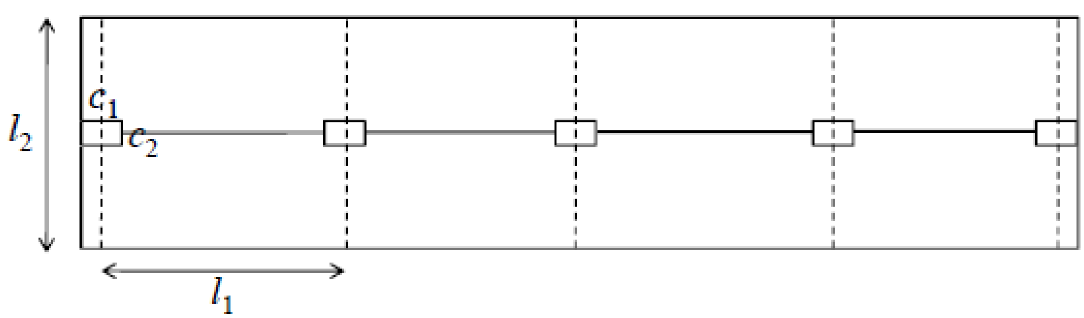


Figure 9: Slab panels with columns

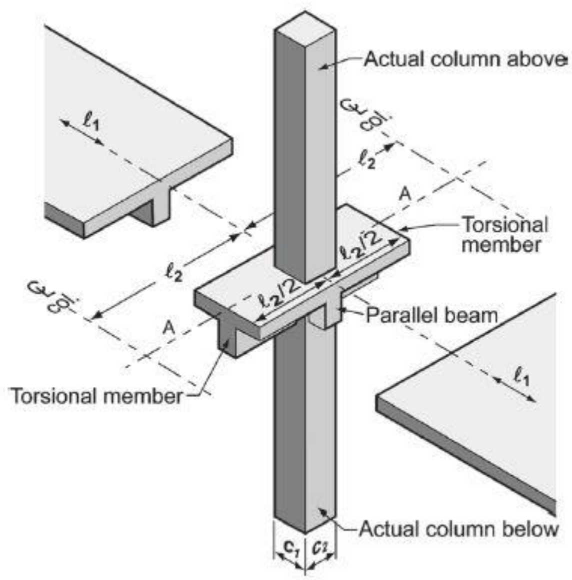
3. Following the determination of factor k and slab-beam moment of inertia I_{sb}, stiffness of slab-beam is determined as follows:

$$\boldsymbol{Ksb} = (\boldsymbol{kEIsb})/(\boldsymbol{l})$$

3.2.3 Stiffness of torsional member (K_t)

1. Torsional members (transverse members) provide moment transfer between the slab-beams and the columns.
2. Assumed to have constant cross-section throughout their length.

$$K_t = \sum \frac{9E_{cs}C}{\ell_2(1 - c_2/\ell_2)^3}$$



3. The constant *C* in equation is calculated by subdividing the cross section into rectangles and carrying out the summation. Where *x* is the shorter side of a rectangle and *y* is the longer side of the torsional member.

$$C = \sum\left[\left(1 - 0.63\frac{x}{y}\right)\frac{x^3y}{3}\right]$$

4. If beams frame into the support in the direction of analysis the torsional analysis, stiffness Kt needs to be increased.

$$K_{ta} = \frac{I_{sb} K_t}{I_s}$$

Where,

Isb = moment of inertia of slab with beam

Is = moment of inertia of slab without beam

3.2.4 Stiffness of actual columns (K_c)

$$k_a = 4.0 \left(\frac{t_a}{t_b} \right)^{0.08} \left(\frac{\ell_c}{\ell_u} \right)^{2.7} \geq 4.0$$

for **ta/tb**=0.4 to 2.2 and ***l*c/*l*u** upto 1.2

a-end=column end near the slab to be analyzed

b-end=column end away from the slab to be analyzed

ta= thickness value at a-end of column

tb=thickness value at b-end of column

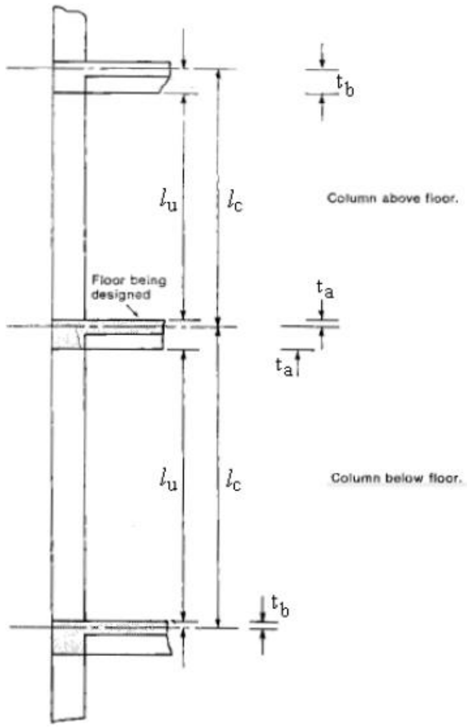


Figure 10: Stiffness of Actual Column

3.2.5 Stiffness of Equivalent column (K_{ec})

Stiffness of equivalent column consists of stiffness of actual columns K_c plus stiffness of torsional members K_t

$$\frac{1}{K_{ec}} = \frac{1}{\sum K_c} + \frac{1}{K_t}$$

3.2.6 Calculation of carry over factors

1. For beam carry over factor is as follows

$$COF = 0.57 \left(\frac{c_1}{\ell_1} \times \frac{c_2}{\ell_2} \right)^{0.02} \alpha^{0.37} \qquad \geq 0.5$$

2. For column carry over factor is as follows:

$$COF_a = 0.5\ \frac{(\ell_c / \ell_u)}{(t_a / t_b)^{0.08}} \geq 0.5$$

l_c = centre to centre height of the columns

l_u = unsupported length of the column

$l_u = l_c - t_a - t_b$

t=vertical distance starting from centerline up to inner end of slab

$$\text{FEM} = m \times q\ \ell_{2w}\ \ell_1^{\,2}$$

Where,

$$m = 0.09 \left(\frac{c_1}{\ell_1} \times \frac{c_2}{\ell_2}\right)^{0.015} \alpha^{0.24} \geq 0.083$$

3.2.6.1 Distribution Factor

1. Joints in the structure were identified and stiffness factors (**K_{sb}, K_{ec}**) are already calculated.
2. Using these stiffness distribution factors (DF) can be calculated from following equation.

$$\boldsymbol{DF = \frac{K}{\sum K}}$$

DF=0 (For Fixed End)

DF=1 (For Pin End)

3.2.7 **Moment Distribution**

Following procedure was adopted for determining end moments on beam and column spans:

1. Fixed end moment are calculated from step no.5
2. Negative FEMs act counter clockwise on the span while positive FEMs act clockwise(Convention).
3. Moment Distribution Process:
   - Find out the moment which is needed to put each joint in equilibrium.
   - Release the joints and divide the counter balancing moments into the connecting span at each joint.
   - Carry these moments in each span over to the other end by multiplying moment with the carry-over factor (COF) for columns and beams respectively.
   - By repeating the sequence of locking and unlocking the joints, it will be found that the moment corrections will diminish since the beam tends to achieve its final deflected shape. When a small enough value for the corrections is obtained, the process of cycling should be stopped with no "carry-over" of the last moments. Each column of FEMs, distributed moments, and carry-over moments should then be added. If this is done correctly, moment equilibrium at the joints will be achieved.

3.2.8 **Correction of moments to the face of supports**

For design, negative moments must be evaluate at the critical sections. These critical sections are defined by ACI 8.11.6.1.

For interior supports, the critical section for negative *Mu* in both middle and column strips is taken at the face of rectilinear supports, but not farther away than $0.175\ell_1$ from the center of column**.**

3.2.9 Maximum Moments

If the slab to be designed meets the requirement of Direct Design Method ,the total design moment in a control panel can be reduced so that the absolute sum of positive moment and average negative moments does not exceed the statical moment $\boldsymbol{Mo}$ACI **8.10.3.2.**

3.2.10 Distribution of Panel Moments in transverse direction

$$\boldsymbol{\alpha}\mathbf{f} = \frac{\mathbf{Ib}}{\mathbf{Is}}$$

Where,

I_b = Moment of inertia of beams section about centroidal axis

I_s = Moment of inertia of slabs

α = ratio of flexural stiffness of a beam section to a slab

$$\boldsymbol{\beta} = \frac{\boldsymbol{C}}{\boldsymbol{2Is}}$$

$$\beta = \frac{torsional\ rigidity\ of\ edge\ beam\ section}{flexural\ rigidity\ of\ slab\ of\ width\ equal\ to\ beam\ span\ length}$$

3.2.11 Distribution of Moments

Distribution of moments along middle strip, column strip and beams

Determine the distribution factors for the positive and negative moments in the longitudinal and transverse directions for each column and middle strips in both interior and exterior panels as follows:

1. For interior panels, use moment factors in Table given belowif l_1>$l_2$then the distribution in long and short directions as follows:

$$M_{01} = (q_u l_2)\frac{l_{n1}^2}{8} \qquad M_{\mathrm{pl}} = 0.35 M_{0\mathrm{l}} \qquad M_{\mathrm{nl}} = 0.65 M_{0\mathrm{l}}$$

$$M_{0\mathrm{s}} = (q_u l_1)\frac{l_{n2}^2}{8} \qquad M_{\mathrm{ps}} = 0.35 M_{0\mathrm{s}} \qquad M_{\mathrm{ns}} = 0.65 M_{0\mathrm{s}}$$

For the distribution of moments in the transverse direction, use Table for column strips.

Table 1: Percentage of Longitudinal Moments in Column strip, Interior plan

		Aspect Ratio, l_2/l_1		
	$\alpha_{f_1} l_2/l_1$	0.5	1.0	2.0
Negative moment at interior support	0	75	75	75
	≥1.0	90	75	45
Positive moment near midspan	0	60	60	60
	≥1.0	90	75	45

The middle strips will resist the portion of the moments not assigned to the column strips.

2. For exterior panels, use moment factors in Table

Table 2: Distribution of Moments in End Plan

	Exterior Edge		Slab with Beams	Slab without Beams between Interior Supports	
	Unrestrained (1)	Fully Restrained (2)	between All Supports (3)	With Edge Beam (4)	Without Edge Beam (5)
Exterior negative factored moment	0	0.65	0.16	0.30	0.26
Positive factored moment	0.63	0.35	0.57	0.50	0.52
Interior negative factored moment	0.75	0.65	0.70	0.70	0.70

For the distribution of moments in the transverse direction, use Table for the column strip.

Table 3: Percentage of Longitudinal Moments in Column strip, Exterior panel

	$\alpha_{f_1} l_2/l_1$	β_t	Aspect Ratio l_2/l_1		
			0.5	1.0	2.0
Negative moment at exterior support	0	0	100	100	100
		≥2.5	75	75	75
	≥1.0	0	100	100	100
		≥2.5	90	75	45
Positive moment near midspan	0		60	60	60
	≥1.0		90	75	45
Negative moment at interior support	0		75	75	75
	≥1.0		90	75	45

The middle strip will resist the balance of the panel moment.

3. In both cases (1) and (2), the beams must resist 85% of the moment in the column strip when $\alpha f1(l2/l1) \geq 1.0$, whereas the ratio varies between 85 and 0% when $\alpha f1(l2/l1)$varies between 1.0 and 0

Table 4: Portion of Column strip Mu in beams

$\alpha_{f1}\ell_2/\ell_1$	Distribution coefficient
0	0
≥1.0	0.85

Note: Linear interpolation shall be made between values shown.

3.2.12 Slab Shear

ACI **6.5.4***Vu* due to gravity loads shall be calculated in accordance with Table

Table 5: Approximate Shears for non-prestressed continuous beams and one way slab

Location	V_u
Exterior face of first interior support	$1.15 w_u \ell_n/2$
Face of all other supports	$w_u \ell_n/2$

Shear stresses in slab are not critical. Shear stresses are calculated at a distance d from the supporting beans because it is a critical section. For exterior face of first interior support shear stresses will be:

$$Vu = 1.15wu(\left(\frac{l2}{2}\right) - \frac{1}{2}beam\ width - d)$$

Shear capacity of the concrete slab section is

$$\emptyset Vc = \emptyset\left(2\sqrt{f'c}\right)bd$$

For design the shear capacity should be greater than shear stresses

$$\emptyset Vc > Vu$$

3.2.13 Reinforcement Limits

(ACI 8.6.1.1) A minimum area of flexural reinforcement, $\boldsymbol{As_{min}}$, shall be provided near the tension face in the direction of the span under consideration in accordance with Table

Table 6: As_{min} for non-prestressed two way slabs

Reinforcement type	f_y, psi	$A_{s,min}$, in.^{2}	
Deformed bars	< 60,000	$0.0020A_g$	
Deformed bars or welded wire reinforcement	≥ 60,000	Greater of:	$\frac{0.0018 \times 60,000}{f_y}A_g$
			$0.0014A_g$

3.2.14 Reinforcement Detailing

Reinforcement can be calculated by formula given below

$$\frac{\boldsymbol{(As)^2(fy)^2}}{\boldsymbol{1.7(f'c)b}} - \boldsymbol{As(fy)d} + \frac{\boldsymbol{Mu}}{\emptyset} = \boldsymbol{0}$$

Calculate the As steel area required per foot by using the design moments **Mu** already calculated in the beams , column strips and middle strips and strength reduction factor $\emptyset = 0.9$ then calculate the spacing between appropriate area bars selected.

Use appropriate area **Av** reinforcement bars and calculate spacing of bars in the slab such that spacing does not exceed the ACI limits of maximum spacing: 18 *in* or twice the thickness of slab thickness, whichever is smaller.

$$s = \frac{12Av}{As}$$

3.3 RC Structure's Layout Optimization

3.3.1 Ant Colony Algorithm

Ant Colony Algorithm is manifestation of discrete mathematics designed to converge combinatorial problems or to opt for the most viable option without evaluating the effectiveness of each choice. It is heuristic in nature and evolves with the advance in iterations.

Fundamentally ACO algorithm mimics the information sharing and manipulation by ant colonies to determine the shortest route to a food source. Ants leave pheromone trails indicating potential food sources while pheromone level or quantity of pheromone deposited on a node is proportional to the suitability of the route under consideration. At first, the ants wander randomly. When an ant finds a source of food, it walks back to the colony leaving "markers" (pheromones) that show the path has food. When other ants come across the markers, they are likely to follow the path with a certain probability. If they do, they then populate the path with their own markers as they bring the food back. As more ants find the path, it gets stronger until there are a couple streams of ants traveling to various food sources near the colony.

Quite similar to the ants' search for shortest food path, we are seeking the most effective layout of a regular RCC structure by modeling a structure in term of nodes, defining the solution space and decreasing or increasing the probability of a node to get picked in the following iteration on the basis of the objective function evaluation. Because the ants drop pheromones every time they bring food, shorter paths are more likely to be stronger, hence optimizing the "solution." In the meantime, some ants are still randomly scouting for closer food sources.

ACO converges when probabilities of effective nodes are considerably higher than the rest of solution space resulting in best nodes to be picked repeatedly until the cost difference between iterations reduces to an insignificant magnitude or following iterations are already preceded by an optimal solution.

ACO developed in our case for RCC structure optimizes layout by following the steps given below:

1. Construction of solution space (definition of ranges, allowable limits to alter layout).
2. Input of typical layout to initiate evaluation on the basis of objective function (cost). Probabilities of different nodes are calculated on the basis of following eq.

$$p_{ij}^{k} = \begin{cases} \dfrac{[\tau_{ij}]^{\alpha}[\eta_{ij}]^{\beta}}{\sum_{l \in N_i^k}[\tau_{il}]^{\alpha}[\eta_{il}]^{\beta}} & \text{if } j \in N_i^k \\ 0 & \text{if } j \notin N_i^k \end{cases}$$

Where,

τ_{ij} is magnitude of pheromone on the trails

η_{ij} is heuristic value, to be specified on the basis of experience

N_i^k is feasible neighborhood

α and β determines the relative influence of Pheromone trails and heuristic value

Table 7 : Parameters for ACO Algorithm

Number of ants	α	B	λ	ρ	τ^1_{min}	τ^1_{max}	τ^2_{min}	τ^2_{max}
2 * 80	2	5	0.5	0.005	4.3	9.7	1.2	4.8

3. Change in layout from previous variable values and reevaluation.
4. Comparison of objective function values computed in different iterations.

5. Pheromone evaporation on each node decreasing probability of each node equally irrespective of effectiveness followed by increase in pheromone level for best layout of an iteration cycle and most-effective layout determined so far.

$$\tau_{ij} \leftarrow (1-\rho)\tau_{ij} \quad \forall (i,j) \in A$$

$$\tau_{ij} \leftarrow \tau_{ij} + \Delta\tau_{ij}^{best}, \quad \forall (i,j) \in A$$

ρ = evaporation rate

6. Random node selection is also introduced to prevent convergence to a local minima or maxima.

3.4 RC Structure’s Element Optimization

The main objective of optimization is to hunt for the best solution using efficient equivalent methods. In this procedure, decision variables depict the quantities to be find out, and a number of decision variables comprising a candidate solution.

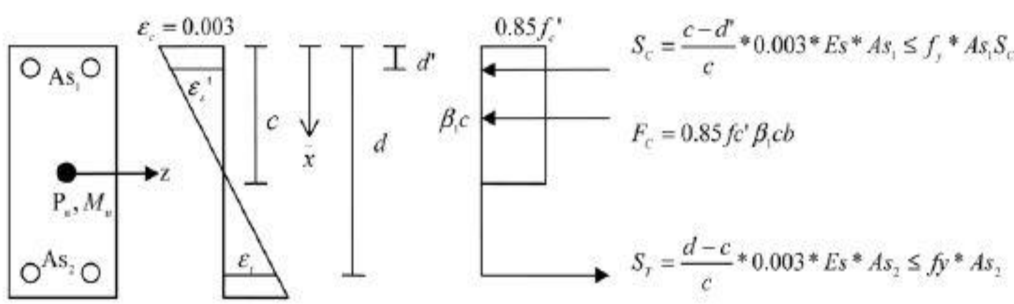


Figure 11: Reinforced Concrete section and Resistive forces

3.4.1 Objective Function

The set of allowable solution which are either maximized or minimized, express the performance criterion, or goals in terms of decision variables. The main objective of the optimization is to reduce the cost of structure by not avoiding the ACI318_14 code's strength and serviceability conditions or limits.

$$Minimize\ F(x) = F_b + F_c$$

$$Where\ F_b = \sum_{i=1}^{N_b} C_c.(V_{it} - V_{is}) + C_s V_{is} \gamma_{is} + C_f.A_{fb}$$

$$And\ F_c = \sum_{i=1}^{N_c} C_c.(V_{it} - V_{is}) + C_s V_{is} \gamma_{is} + C_f.A_{fc}$$

$$Subject\ to\ C \leq 0\ where\ C = \sum_{i=1}^{n} c_i$$

Where,

F(x) = Objective function which represents the total cost of the frame in PKR

F_b = Total cost of Beams in a frame structure

F_c = Total cost of Columns in a frame structure

N_B = Number of Beams in a frame structure

N_C = Total cost of Columns in a frame structure

C_c = Cost of concrete

C_s = Cost of steel

C_f = Cost of formwork

V_{it} = Total volume of members

V_{is} = Volume of steel reinforcement in members

A_f = Formwork area

γ_s = Density (Weight per unit volume)

C = Penalty (constraint violation) function

c_i = violation function of a specific constraint

n = number of constraints for a given frame

3.4.2 **Constraints**

3.4.2.1 Beam Constraints

Beams constraints administer the moment capacity, reinforcement limitations, adequate shear strength and spacing as well. All of these constraints are explained below.

1. **Moment Strength**

A beam must have adequate flexural strength $\emptyset \boldsymbol{Mn}$ that is able to resist the applied moments $\boldsymbol{Mu}$, to be considered as an adequate beam. If condition is not specified, a constraint is given.

$$ca = \frac{Mu - \emptyset Mn}{\emptyset Mn} \geq 0$$

2. **Minimum Reinforcement Area**

That is the area of reinforcement which must be larger than the minimum reinforcement area which is specified in ACI code.

$$cb = \frac{Ast,min - Ast}{Ast,min} \geq 0$$

$$A_{s,min} = \frac{0.25\sqrt{f_c'}}{f_y} b_w d \geq \frac{1.4 b_w d}{f_y}$$

3. **Minimum Ductility for Reinforcement**

That is a check to ensure that Beam must fail in Tension not the compressio , means beams is in tension-controlled region. According to ACI code strain in extreme steel layer $\boldsymbol{\varepsilon_t}$ must exceed 0.004:

$$cc = \frac{0.004 - \boldsymbol{\varepsilon t}}{0.004} \geq 0$$

4. **Minimum Bar Spacing**

To avoid the segregation and the smooth flow of concrete, there should be adequate bar spacing according to ACI code minimum spacing $\boldsymbol{S_{min}}$.

$$cd = \frac{Smin - S}{Smin} \geq 0$$

$$S_{min} = larger\ of\left(25mm, d_b, \frac{4}{3} Max.Agg.Size\right)$$

5. **Deflection Characteristics**

The height of reinforced beam section is one method to limit deflections in reinforced concrete beams not part of a moment resisting frame.

$$ce = \frac{hmin - h}{hmin} \geq 0$$

3.4.2.2 Column constraints

1. **Axial Strength**

The column's axial strength $\emptyset Pn$ must be greater than the applied factored load Pu , therefore the axial strength constraint can be calculated as given below:

$$c1 = \frac{Pu - \emptyset Pn}{\emptyset Pn} \geq 0$$

2. **Moment Strength**

A column must have adequate bending strength $\emptyset Mn$ to resist the applied factored bending moment Mu. Therefore, the constraint of flexural strength can be calculated as given below:

$$c2 = \frac{Mu - \emptyset Mn}{\emptyset Mn} \geq 0$$

3. **Shear Strength**

A column must have adequate shear strength $\emptyset Vn$ to resist the applied factored shear forceVu. Therefore, constraint for shear strength can be calculated as below:

$$c3 = \frac{Vu - \emptyset Vn}{\emptyset Vn} \geq 0$$

4. **Minimum Reinforcement Ratio**

Limiting reinforcement ratio is 1%. So, we cannot take lower value than that. Therefore, constraint for minimum reinforcement can be calculated as given below:

$$c4 = \frac{0.001 - \rho}{0.001} \geq 0$$

5. **Maximum Reinforcement Ratio**

The limiting value for maximum reinforcement ratio is 8%. So, we cannot take value larger than the 8%. Hence, constraint for maximum reinforcement ratio can be computed using:

$$c5 = \frac{\rho - 0.008}{0.008} \geq 0$$

6. **Dimension compatibility between columns and beams**

In most engineering practices it is considered good to have the column dimension in top stories equal or lower than the base columns. The constraints for dimension compatibility can be calculated by using:

$$c6 = \frac{b, top - b, bottom}{b, bottom} \geq 0$$

$$c7 = \frac{h, top - h, bottom}{h, bottom} \geq 0$$

7. **Minimum Bar Spacing**

To avoid the segregation and the smooth flow of concrete, there should be adequate bar spacing according to ACI code minimum spacing S_{min}. Hence, constraint for sapcing can be computed by using:

$$c8 = \frac{Smin - S}{Smin} \geq 0$$

3.5 Design of Reinforced Concrete Frame Elements

This section discusses the design of frame elements according to ACI provisions. The design issues related to beams and columns are discussed related to shear force and bending moments.

3.5.1 **Design Concept**

According to ACI the reduced design capacity of the member should be greater than factored applied force. This concept is described by this equation:

$$\emptyset Rn \geq Ru$$

Where,

$\emptyset$ = strength reduction factor

R_n= nominal resistance of a reinforced concrete element

R_u=applied ultimate external load

Ultimate external load should be selected from load combinations suggested in ACI Code. For this research work only two primary loading combinations are considered.

1. R_u = 1.4D primary load is dead load
2. R_u= 1.2D + 1.6L primary load is live load

3.5.2 **Beam Analysis and Design**

Gravity loads are applied on the beam which results in compression and tension stresses. These stresses create a couple moment which is to be resisted by bending moment of the section.

Since concrete is weak in tension it is assumed that all tensile stresses are resisted by the tension bars and compression stresses are taken by concrete.

For the analysis of beams in flexure, the following assumptions are made by the ACI code section (10.2), which are as follows:

1. Strain in reinforcement and concrete is considered directly proportional to the distance from the neutral axis.
2. Maximum strain at extreme concrete compression zone is considered to be 0.003.
3. Stress in reinforcement less than fy should be taken as Es (Elasticity of Steel) times steel strain.
4. In axial and flexural calculations of reinforced concreteTensile strength of concrete should be neglected.
5. The relationship of concrete strain and concrete compressive stress shall be considered to be trapezoidal, parabolic, rectangular or any shape that results in an accurate guess of strength.

Based on the above given assumptions there are two forces acting on beam section due to moment.

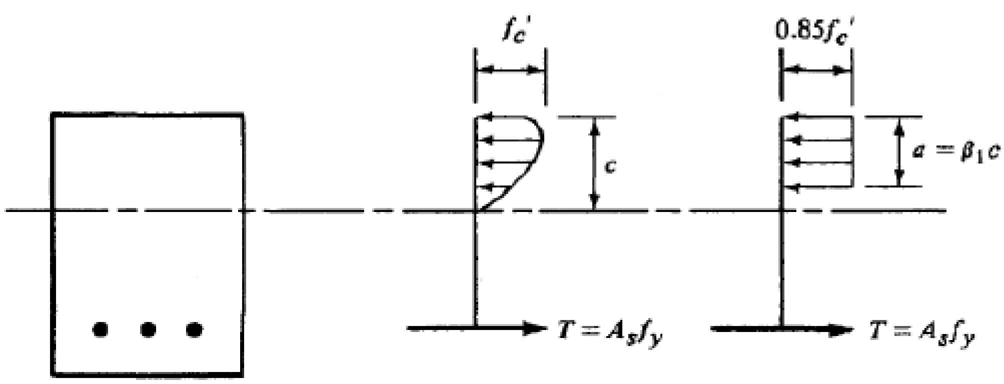


Figure 12: Resistive forces acting on beam

The shape of the compression block is parabolic which can be exchanged by an equivalent rectangular block called Whitney's Rectangular Stress Distribution as shown in above figure.

This rectangular stress block's intensity is *0.85*f'cand depth of *a*, that is related to the depth of the neutral axis *c* according to ACI section **22.2.2.4.3** as follows:

Table 8: Values of β for equivalent rectangular concrete stress distribution

f_c', psi	β_1	
$2500 \le f_c' \le 4000$	0.85	(a)
$4000 < f_c' < 8000$	$0.85 - \frac{0.05(f_c' - 4000)}{1000}$	(b)
$f_c' \ge 8000$	0.65	(c)

$$a = \beta 1 * c$$

$$a = \frac{As * fy}{0.85 b f'c}$$

Where,

f'c= concrete's compressive strength (psi),

a = depth of compression block (in)

b = beam's width (in)

fy= yielding strength of steel (psi).

For equilibrium of moments:

$$\emptyset Mn = Mu = \emptyset Asfy(d - \frac{As * fy}{1.7 b f'c})$$

Where

∅= Strength reduction factor.

Mn= Nominal flexure capacity of beam (Kip-in),

As = Total area of reinforcement in tension (in^2)

d = distance from the extreme compression fiber to the center of tension reinforcement.

Beam cross-section can be tension-controlled as well as compression-controlled.

When the tensile strain in the extreme fiber of steel is 0.002 and strain in the extreme fiber of concrete reaches 0.003 then beams will be tension-controlled. Strength reduction factor will be $\emptyset = 0.65$.

Similarly, when the strain in the steel is 0.005 and concrete strain is 0.003 then beam will be compression controlled. In this case there will be warning of failure by excessive deflection and cracking might be occurred. Strength reduction factor will be$\emptyset = 0.9$.

Section between these two extreme cases strength reduction factor will be calculated by linear interpolation. Thus can be related to extreme tension strain as shown by this equation

$$\emptyset = 0.65 + \frac{(\varepsilon t - 0.002)0.25}{(0.005 - 0.002)}$$

3.5.2.1 Deflection Control

There are two methods which are used in ACI code for limiting deflections in beams not attached to any partitions.

1. First method is based on minimum thickness calculated by span length.

Table 9: Minimum depth of nonprestressed beams

Support condition	Minimum h[1]
Simply supported	$\ell/16$
One end continuous	$\ell/18.5$
Both ends continuous	$\ell/21$
Cantilever	$\ell/8$

2. Second method is by directly applying limits on deflection.

Table 10: Deflection Limits

Type of member	Deflection to be considered	Deflection limitation
Flat roofs not supporting or attached to non-structural elements likely to be damaged by large deflections	Immediate deflection due to live load L	$L/180$
Floors not supporting or attached to non-structural elements likely to be damaged by large deflections	Immediate deflection due to live load L	$L/360$
Roof or floor construction supporting or attached to non-structural elements likely to be damaged by large deflections	That part of the total deflection occurring after attachment of nonstructural elements (sum of the long-term deflection due to all sustained load and the immediate deflection due to any additional live load	$L/480$
Roof or floor construction supporting or attached to non-structural elements not likely to be damaged by large deflections		$L/240$

3.5.3 Column Analysis and Design under Axial loads and Bending

Columns are members used to support axial compressive loads. Columns can be axially loaded, uniaxial and bi-axial columns.

3.5.3.1 Axially Loaded Columns

Design Equations

To reduce the loading capacity P_o of column, the ACI Code specifies that the maximum nominal load, P_o, should be multiplied by a factor equal to 0.8 for tied columns

$$Pu = \emptyset Pn = \emptyset 0.8[0.85f'c(Ag - Ast) + Astfy]$$

Or

$$Pu = \emptyset Pn = \emptyset 0.8Ag[0.85f'c + \rho(fy - 0.85f'c)]$$

Ag = gross concrete area

Ast= total steel compressive area

φ = 0.65 for tied columns

For design of axially loaded column following steps are considered:

1. Gross area of column is calculated by this equation. Percentage of steel $\boldsymbol{\rho}$is obtained by optimization tool of Matlab such as *fmincon*. $\boldsymbol{\rho}$ varies between 1% to 8% of the gross area of column.

$$Ag = \frac{Pu}{0.85f'c + \rho fy}$$

2. Then area of steel is calculated as percentage is multiplied by gross area of column

$$As = \rho * Ag$$

3. Then ultimate load is calculated by this equation

$$Pu = \emptyset 0.8Ag[0.85f'c + \boldsymbol{\rho}(fy - 0.85f'c)]$$

4. Finally column section adequacy is checked by this equation such that section capacity should be greater than applied axial load

$$Pu \geq \emptyset Pn$$

3.5.3.2 Uni-axially loaded Columns

There are following steps which we adopted in design of uni-axially loaded columns. Since uni-axially columns take compression load. Therefore, these columns are always designed as compression controlled.

1. Uniaxially design approach is adopted once eccentricity is greater than 10% of height of column section. i.e. e > 0.1h

Where $$e = \left(\frac{Mu}{P}\right)$$

2. First of all we approximate the gross area of column by following equation

$$Ag = \frac{Pu}{0.4f'c}$$

3. Then dimensions are calculated by taking the square root of gross area

$$h = \sqrt{Ag}$$

4. After finding dimensions of cross-section , slenderness of column is found by following equation:

$$\frac{L}{h} < 12$$

L=length of column between bottom and upper support

h=width of column

if above given condition is satisfied then the column is being designed as short column i.e. column will fail by crushing of concrete not by buckling:

Then *a* is calculated which is the rectangular stress block's depth can be calculated by

$$Mu = 0.85f'cab\left(\frac{h}{2} - \frac{a}{2}\right) + Asfy\left(d - \frac{h}{2}\right) + Asfy(\frac{h}{2} - \frac{a}{2})$$

Where c is calculated by equation

$$c = (\frac{a}{\beta})$$

5. Finally check for compression controlled column is done such that following condition satisfied:

$$\frac{c}{dt} \geq \frac{87}{147}$$

6. if the column is compression controlled than section capacity is checked such that:

$$Mu < \emptyset Mn \quad \text{or} \quad Pu < \emptyset Pn$$

3.5.3.3 Bi-axially loaded Columns

Following method is used to design bi-axially loaded columns

The equivalent eccentricity method

Bi-axial eccentricities e_x and e_y are replaced by an equivalent eccentricity e_{ox}. Then column is designed as uni-axial column with uni-axial bending and axial load. M_{ux} and M_{uy} are related to e_x and e_y as follows:

$$M_{uy} = P_u e_x M_{ux} = P_u e_y$$

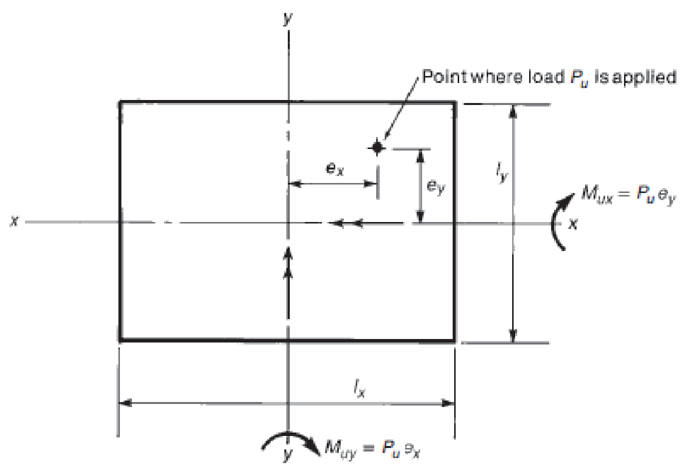


Figure 13: Bi-axially loaded columns

1. If the given condition satisfies such that

$$\frac{ex}{lx} \geq \frac{ey}{ly}$$

then the column can be designed for P_u and a factored moment $M_{oy}=P_u e_{ox}$ as uni-axial columns where

$$\mathbf{e_{ox}=e_x + (\alpha e_y l_x)/l_y}$$

2. If the condition is

$$\frac{ey}{ly} \geq \frac{ex}{lx}$$

then the column can be designed for P_u and a factored moment $M_{ox}=P_u e_{oy}$ where

$$e_{oy}=e_y+(\alpha e_x l_x)/l_y$$

3. Condition is checked for α

where for $P_u/f'_c A_g \leq 0.4$,

$$\alpha = \left(0.5 + \frac{P_u}{f'_c A_g}\right)\frac{f_y + 40{,}000}{100{,}000} \geq 0.6$$

and for $P_u/f'_c A_g > 0.4$,

$$\alpha = \left(1.3 - \frac{P_u}{f'_c A_g}\right)\frac{f_y + 40{,}000}{100{,}000} \geq 0.5$$

4. There is limitation in bi-axially loaded column such that

$$\frac{lx}{ly} = 0.5 \text{ to } 2$$

Chapter 4

RESULTS AND COMPARISON

4.1 Results

In this Chapter comparison of cost of different elements of frame structure is calculated by using book of "Structural Concrete, Theory and Design" by M. Nadim Hassoun and Akhtem Al. Manaseer and "Reinforced Concrete" by Edward G. Nawy and other solution from our designed software.

At the end the summary sheet of cost benefit is given, which gives a complete view of Cost which we can save by using this optimized design while satisfying all the limitations and considerations of the ACI code.

First Different components are designed and their comparison is given and in the last of the chapter layout optimization's results are given which will be compared with the existing squash court grey structures' cost.

4.1.1 **Columns**

The comparison of the design and cost of an axially column is given below. Book's example is given below and then the same column is designed with the help of our software (SMAF Optimized).

Example 9.1: Analysis of an Axially Loaded Non-Slender Rectangular Tied Column

A non-slender tied column is subjected to axial load only. It has the geometry shown in Figure 9.6a and is reinforced with three No. 9 bars (28.6-mm diameter) on each of the two faces parallel to the x axis of bending. Calculate the maximum nominal axial load strength $P_{n(max)}$. Given:

$$f'_c = 4000 \text{ psi } (27.6 \text{ MPa})$$

$$f_y = 60{,}000 \text{ psi } (414 \text{ MPa})$$

Solution: $A_s = A'_s = 3$ in.^{2}. Therefore, $A_{st} = 6$ in.^{2}. Using Eq. 9.2 yields

$$P_{n(max)} = 0.8[0.85 \times 4000[(12 \times 20) - 6] + 6 \times 60{,}000]$$

$$= 924{,}480 \text{ lb } (4110 \text{ kN})$$

Assuming column height = 12'

Cost of this column = 20786 Rs.

Cost of this optimized column = 18411 Rs.

Example 10.2

Design a square tied column to support an axial dead load of 400 K and a live load of 232 K using $f'_c = 5$ ksi, $f_y = 60$ ksi, and a steel ratio of about 5%. Design the necessary ties.

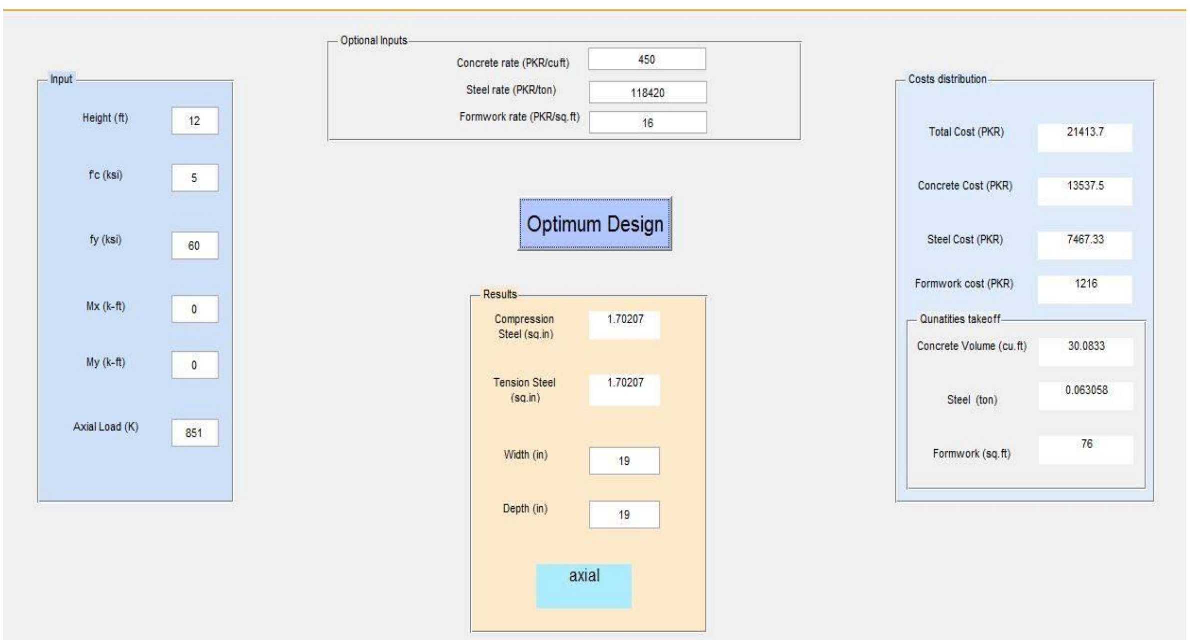
Solution

1. Calculate $P_u = 1.2P_D + 1.6P_L = 1.2(400) + 1.6(232) = 851$ K. Using Eq. 10.10, $P_u = 851 = 0.65(0.8)A_g[0.85 \times 5 + 0.05(60 - 0.8 \times 5)]$, $A_g = 232.5$ in.^{2}, and column side = 15.25 in., so use 16 in. (Actual $A_g = 256$ in.^{2}.)
2. Because a larger section is adopted, the steel percentage may be reduced by using $A_g = 256$ in.^{2} in Eq. 10.8:

$$851 = 0.65(0.8)[0.85 \times 5 \times 256 + A_{st}(60 - 0.85 \times 5)]$$

$$A_{st} = 9.84 \text{ in.}^2$$

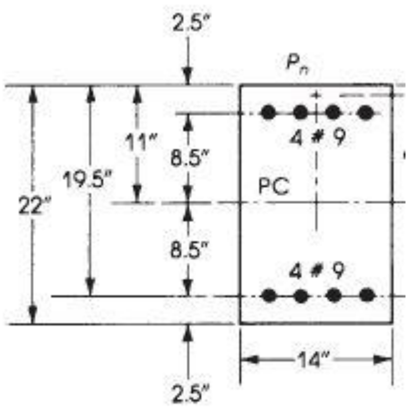
Cost of this column = 29971 Rs.



Cost of optimized column = 21414 Rs.

Example 11.4

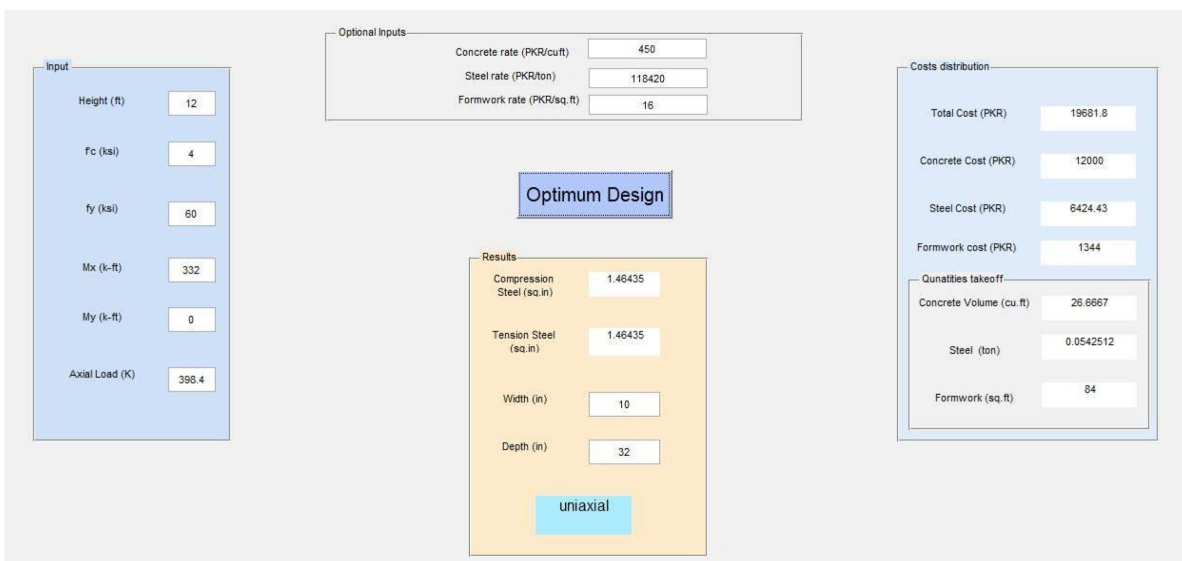
Determine the nominal compressive strength, P_n, for the section given in Example 11.2 if $e = 10$ in.



$$\phi P_n = 0.65(612.9) = 398.4 \text{ K}$$

$$\phi M_n = 0.65(510.8) = 332 \text{ K} \cdot \text{ft}$$

Cost of this column = 27529 Rs.



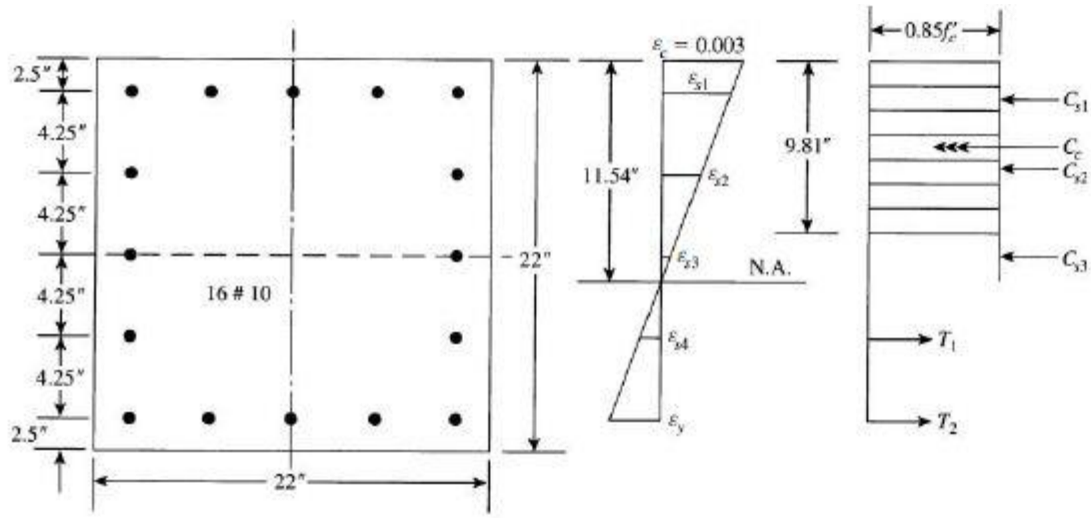
Cost of optimized column = 19682 Rs.

Example 11.7

Determine the balanced load, P_b moment, M_b, and e_b for the section shown in Fig. 11.13. Use $f_c' = 4$ ksi and $f_y = 60$ ksi.

Solution

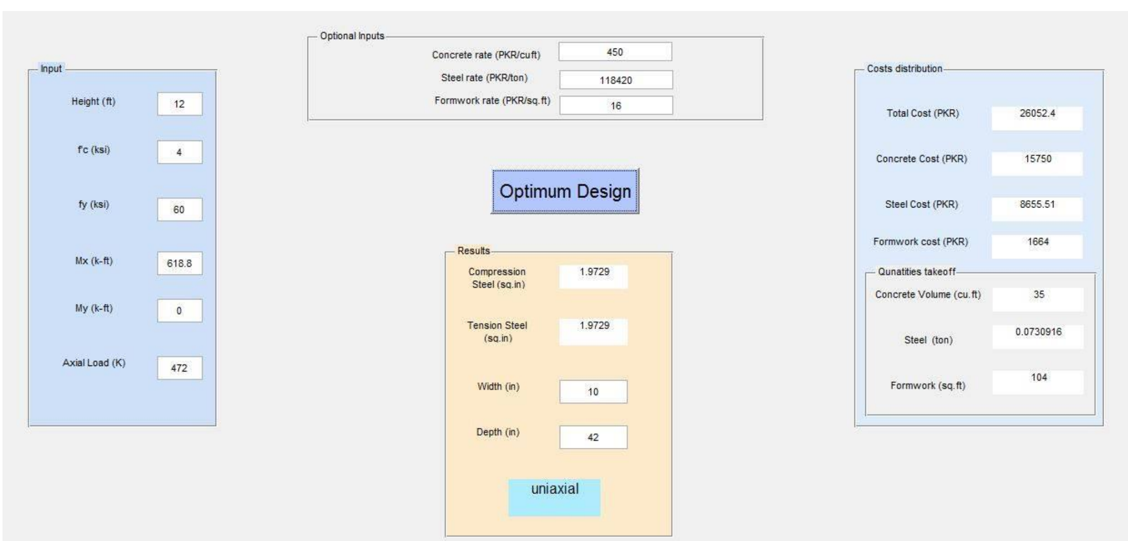
The balanced section is similar to Example 11.2. Given: $b = h = 22$ in., $d = 19.5$ in., $d' = 2.5$ in., $A_s = A_s' = 6.35$ in.^{2} (five no. 10 bars), and six no. 10 side bars (three on each side).



Determine ϕ: For a balanced section, $\varepsilon_t = \varepsilon_y = 0.002, \phi = 0.65,$

$$\phi P_b = 0.65 P_b = 472 \text{ K} \quad \text{and} \quad \phi M_b = 0.65 M_b = 618.8 \text{ K} \cdot \text{ft}$$

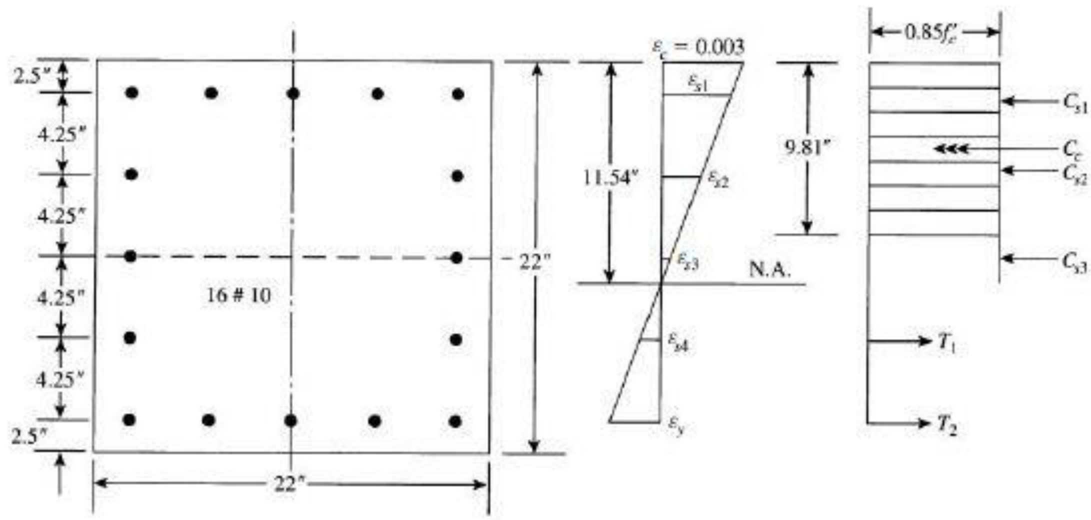
Cost of this column = 43572 Rs.



Cost of optimized column = 26053 Rs.

Example 11.8

Repeat the previous example when $e = 6.0$ in.



Since $\varepsilon_t < 0.002$, then $\phi = 0.65$.

$$\phi P_n = 0.65(1459) = 948.3 \text{ K}$$

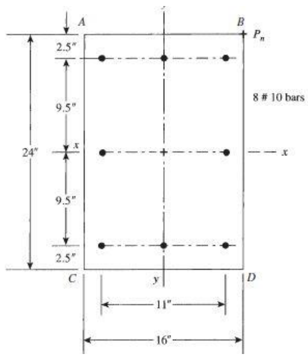
$$\phi M_n = 0.65(729.5) = 474 \text{ K} \cdot \text{ft}$$

Cost of this column = 43572 Rs.

Cost of optimized Column = 30130 Rs.

Example 11.19

The section of a short tied column is 16 × 24 in. and is reinforced with eight no. 10 bars distributed as shown in Fig. 11.29. Determine the design load on the section ϕP_n if it acts at $e_x = 8$ in. and $e_y = 12$ in. Use $f'_c = 5$ ksi, $f_y = 60$ ksi, and the Bresler reciprocal equation.

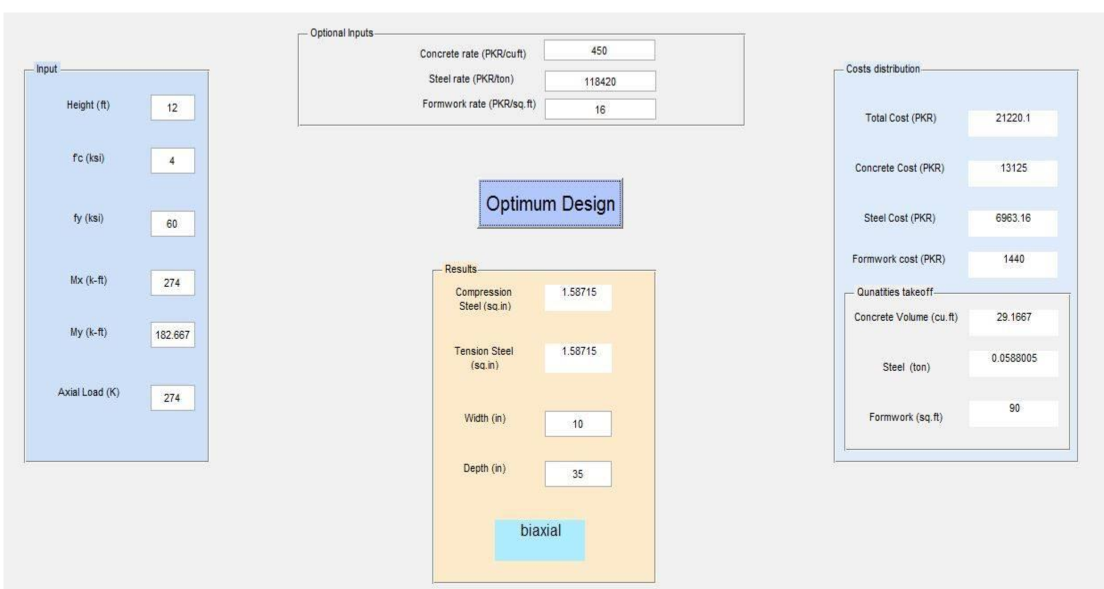


Using the Bresler equation (Eq. 11.31), multiply by 100:

$$\frac{100}{P_u} = \frac{100}{476.2} + \frac{100}{444.5} - \frac{100}{1429} = 0.365$$

$$P_u = 274 \text{ K} \quad \text{and} \quad P_n = \frac{P_u}{0.65} = 421.5 \text{ K}$$

Cost of this column = 28603 Rs.



Cost of optimized column = 21220 Rs.

Table 11: Cost Comparison of columns

Sr.No	Type of Column	Typical Design cost	Optimized Design cost	% Cost Benefit
1	**Axial**			
9.1		20786	18411	11.43%
10.2		29971	21414	28.55%
2	**Uniaxial**			
11.4		27529	19682	28.50%
11.7		43572	26052	40.20%
11.8		43572	30130	30.85%
3	**Bi-axial**			
11.19		28603	21220	23.92%

4.1.2 **Beams**

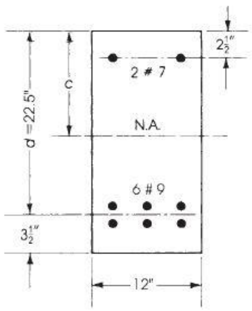
4.1.2.1 Rectangular Beams

The comparison of the design and cost of a rectangular beam is given below. Book's example is given below and then the same column is designed with the help of our software (SMAF Optimized).

Simply supported beam

Example 3.9

A rectangular beam has a width of 12 in. and an effective depth of $d = 22.5$ in. to the centroid of tension steel bars. Tension reinforcement consists of six no. 9 bars in two rows; compression reinforcement consists of two no. 7 bars placed as shown in Fig. 3.26. Calculate the design moment strength of the beam if $f'_c = 4$ ksi and $f_y = 60$ ksi.

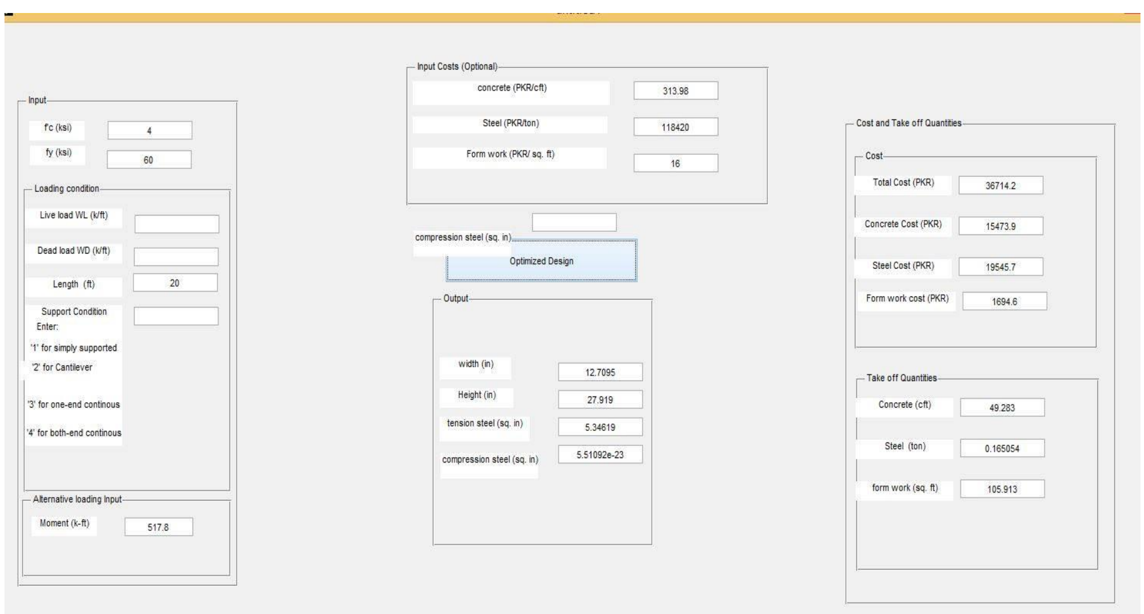


$$\phi M_n = (0.9)\left[4.8 \times 60\left(22.5 - \frac{7.06}{2}\right) + 1.2 \times 60(22.5 - 2.5)\right]$$

$$= 6213\ \text{K} \cdot \text{in.} = 517.8\ \text{K} \cdot \text{ft}$$

Assume span of the beam = 20ft

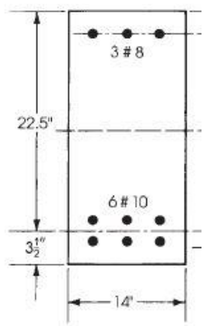
Cost of this beam = 42619 Rs.



Cost of optimized Beam = 36714 Rs.

Example 3.10

Determine the design moment strength of the section shown in Fig. 3.27 using $f'_c = 5$ ksi, $f_y = 60$ ksi, $A'_s = 2.37$ in.^{2} (three no. 8 bars), and $A_s = 7.62$ in.^{2} (six no. 10 bars).

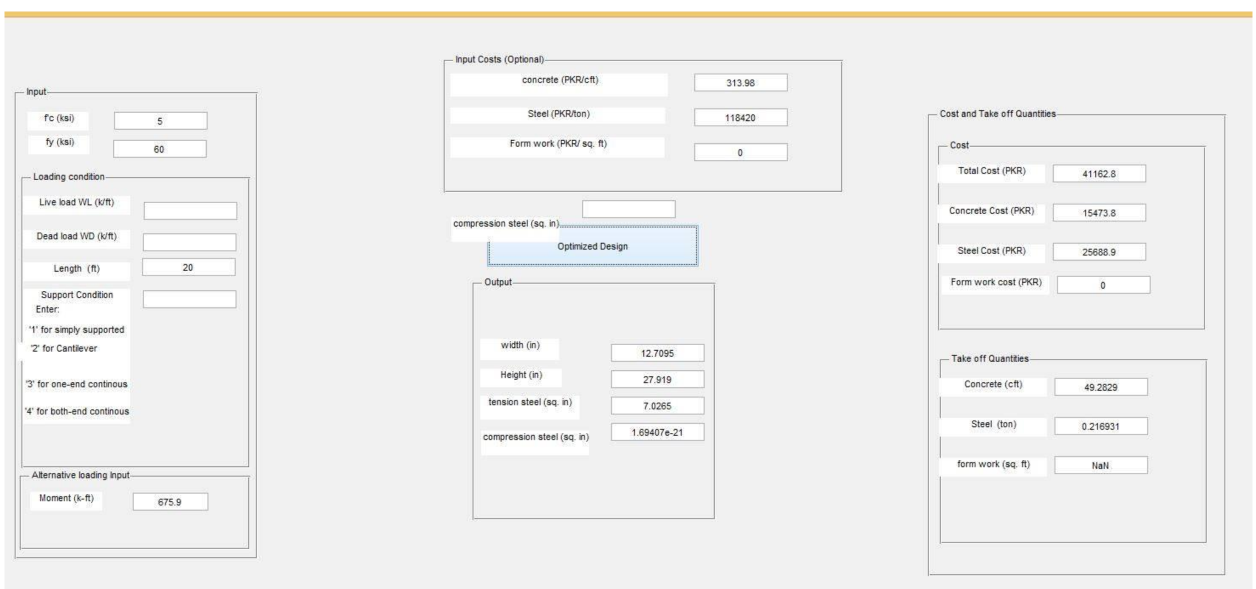


To calculate ϕM_n, take moments about the tension steel A_s:

$$\phi M_n = \phi\left[C_c\left(d - \tfrac{1}{2}a\right) + C_s(d - d')\right] = 0.9[333.2(22.5 - 2.8) + 122.40(22.5 - 2.5)]$$

$$= 8110.8 \text{ K} \cdot \text{in.} = 675.9 \text{ K} \cdot \text{ft}$$

Cost of this beam = 56129 Rs.

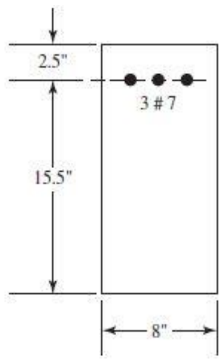


Cost of optimized beam = 41163 Rs.

4.1.2.2 Cantilever Beam

Example 3.4

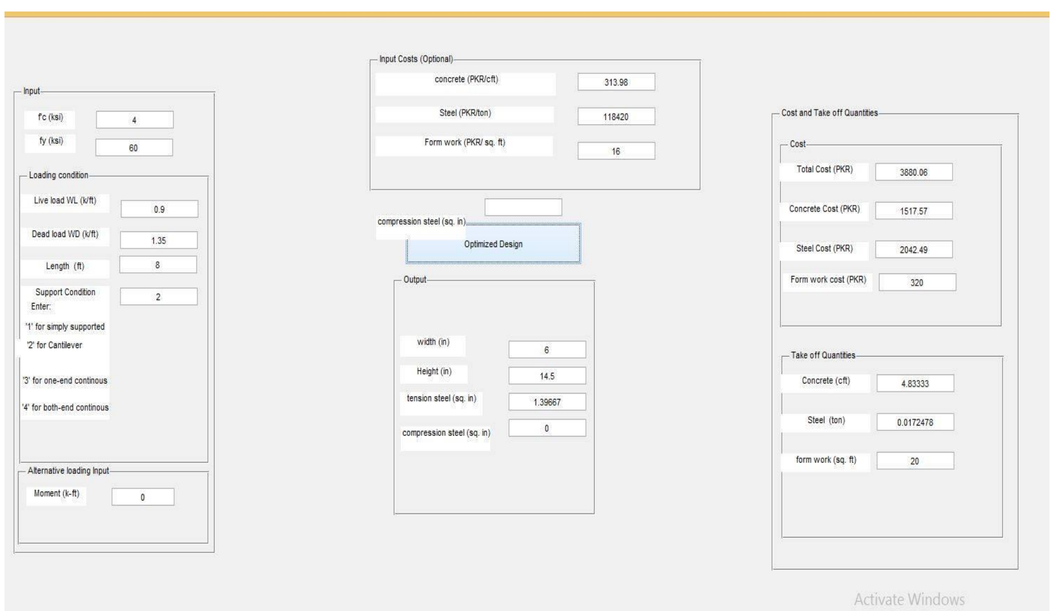
An 8-ft-span cantilever beam has a rectangular section and reinforcement as shown in Fig. 3.17. The beam carries a dead load, including its own weight, of 1.5 K/ft and a live load of 0.9 K/ft. Using $f_c' = 4$ ksi and $f_y = 60$ ksi, check if the beam is safe to carry the above loads.



Calculate ϕM_n:

$$\phi M_n = \phi A_s f_y \left(d - \tfrac{1}{2}a\right)$$
$$= 0.9(1.8)(60)\left(15.5 - \frac{3.97}{2}\right) = 1312 \text{ K} \cdot \text{in.}$$

Cost of this beam = 5413 Rs.



Optimized cost = 3880 Rs.

Table 12 : Cost comparison of Beams

Sr.No	Type of Beam	Typical Design cost	Optimized Design cost	% Cost Benefit
1	**Rectangular**			
3.9		42619	36714	13.86%
3.10		56129	41163	26.66%
2	**Cantilever**			
3.4		5413	3880	28.32%

4.1.3 Layout Optimization

For layout optimization we took an example of existing structure (NUST Squash Court).We calculated the cost of a portion of a squash court which was regular and that cost includes only the cost of steel, concrete and formwork in frame structure.

The comparison of the cost is given below:

Plan of Squash court's part is given below.

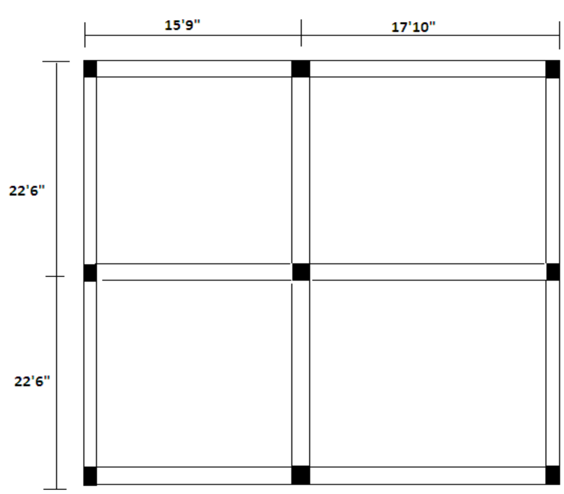


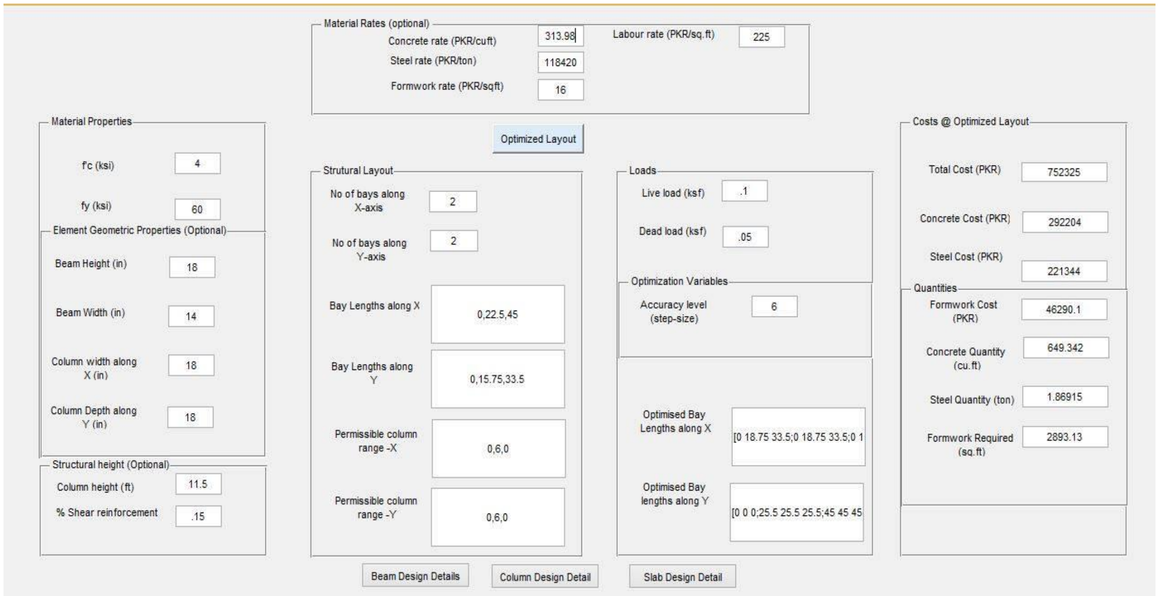
Figure 14: Plan view of frame of squash court

Table 13: BOQ of Squash court

Squash Court NUST					
Sr. NO.	Description	Unit	Quantity	Rate/unit	Amount/Rs
1	**Beams**				
	Cost of Concrete (f'c = 4000psi)	cft	451.5	313.98	141762
	Cost of Steel (fy = 60000psi)	ton	1.71	118420	202498
	Cost of Formwork	sft	1158.75	16	18540
2	**Columns**				
	Cost of Concrete (f'c = 4000psi)	cft	250.13	313.98	78536
	Cost of Steel	ton	0.878	118420	103973
	Cost of Formwork	sft	644	16	10304
3	**Slabs**				
	Cost of Concrete (f'c = 4000psi)	cft	760	313.98	238625
	Cost of Steel (fy = 60000psi)	ton	2.326	118420	275445
	Cost of Formwork	sft	1520.4	16	24327
	Total Cost				**1094010/-**

Cost of this given frame = 1,094,010 Rs.

Now, we will design it with our optimized software and permissible value of 4ft is given, which means column can move max 4ft right or left to give an optimized layout.



Cost of optimized structure = 752,325 Rs.

Table 14: Cost comparison of frame

Description	Typical design cost	Optimized Design cost	% cost benefit
Squash court	1,094,010	752,325	31.2%

Chapter 5

CONCLUSION AND RECOMMENDATIONS

5.1 Commercialization

It is the process of introducing a new product or production method into the market. A project or product is considered feasible if it has the potential to be commercialized. SMAF is a distinctive project and mainly based on research and have decreased the gap between architecture and structural engineer for concrete design. Architecture gives constraints on frame for architectural and aesthetic look while structural engineer try to have the safe and optimized design. SMAF Optimized group is going to sign a patent for launch and commercialization of its optimized software. Steps are taken to obtain copy rights of its product. NUST provides the professional expertise required to launch a product in the market through its Technology Incubation Centre (TIC) and the Centre for Innovation and Entrepreneurship (CIE). We will enlist their help in this regard and launch the product with the help of marketing professionals. The procedure for registering it as an intellectual property has been started with the Intellectual Property Organization, Government of Pakistan (IPO Pakistan). The product will also be patented with the International and US Patent Offices with the help of NUST‘s own Intellectual Property Office (IPO-NUST). Structural engineers will be attracted to use this software because of having cost effective design by this software.

5.2 Conclusions

Cost optimization of RCC structures requires algorithms with better convergence to provide solutions in adequate time to be practically viable. Moreover, the reduced cost should be lesser enough to cause the difference. Our model produced considerably accurate design in appropriate time. The cost savings were about 20-30% for individual elements and 15-20% for layout optimization. These are significant percentages to focus our attention on cost optimization in structural designs so that finances can be reduced or devoted to enhance safety. Also optimized quantity reduces weight of structure which is major source of loads requiring lesser member capacities.

This model is ideal to be used in preliminary design phase reducing the communication gap between architect and engineer. It is developed keeping in view ease of use and flexibility. It is one of the few models which depends on lesser input values and can be constrained when desired. Thus, it can be deduced that better results can be produced if more room is available to consider different options.

5.3 Recommendations

With technological advancement the computational capacity is increasing as well as more sophisticated computational methods are in the limelight. Also complex architectural design requires multi-objective optimization approaches considering all the aspects which may add to cost or affect the serviceability and strength of structures.

5.3.1 Non-regular layout

This research can be further expanded to non-regular RCC structures to generalize floor system and include column with offset.

5.3.2 Other Structural Elements

This study is limited to basic structural elements, to explore practical solutions other elements like foundation, staircase, shear walls etc. can also be included.

5.3.3 **Lateral Loads**

Lateral loads are significant in high rise building, so to encapsulate such buildings lateral load analysis can be included to optimize design.

5.3.4 **Shear Design**

Flexural and axial designs are considered in this study it could be expanded to include shear and torsion designs.

5.3.5 **Joint Detailing**

Joint detailing may affect overall design optimization thus a foolproof structural model should include joint detailing also.

5.3.6 **Uniformity**

Uniformity is key factor to influence construction cost , economical designs with higher design cost may not be overall cost-effective. So uniformity constraints can also be introduced in future models.

Chapter 6

USER GUIDE

Copyright information

Trademark

6.1 Intended audience

This document is intended to be used by system designers for cost effective design of regular buildings. This document will be used along with SMAF software for user help. The configuration software is going to be available on internet soon.

6.2 System requirements

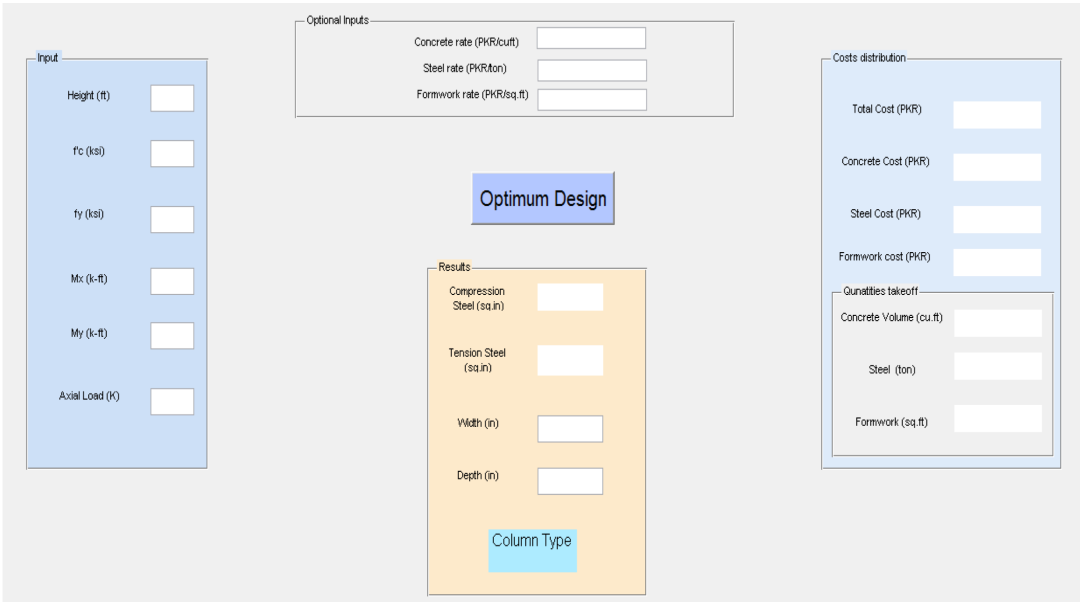
Any current operating system windows (xp,7,8,10) with installed MATLAB on it.

Any version of MATLAB can be user for SMAF GUI.

6.3 Overview of the Software

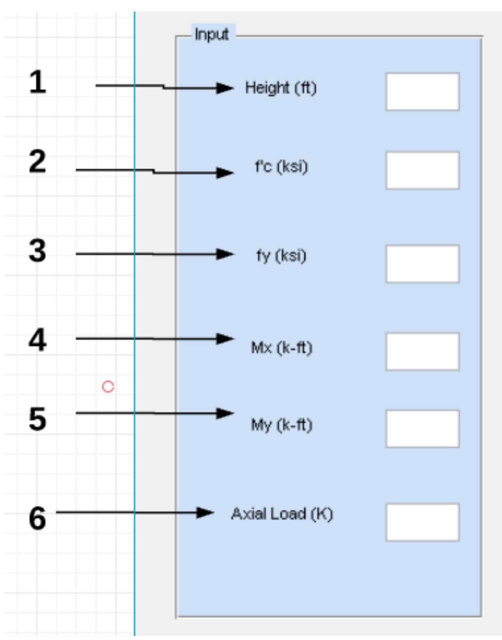
Cost effective design of the regular reinforced concrete building is done by this software. As much as 20 to 30 percent cost is saved by designing by this software. It is very much user friendly. It can be used by both architecture and structural engineer to solve the conflicts between them so that design is optimized as well as can have user constraints on cross-section of frame elements as proposed by architecture. It takes user defined loads and material properties as input and gives the steel required as tension and compression and the cross-section of members of frames. By quantity take-off material quantities and its cost are find for optimized design.

6.4 Using the SMAF Optimized Column software



This is the GRAPHICAL USER INTERFACE (GUI) of Column module. It contains many components which are user friendly takes input and gives output to the user.

6.4.1 Input



1→ is the height of column in feet,

2→is the compressive strength of column in ksi,

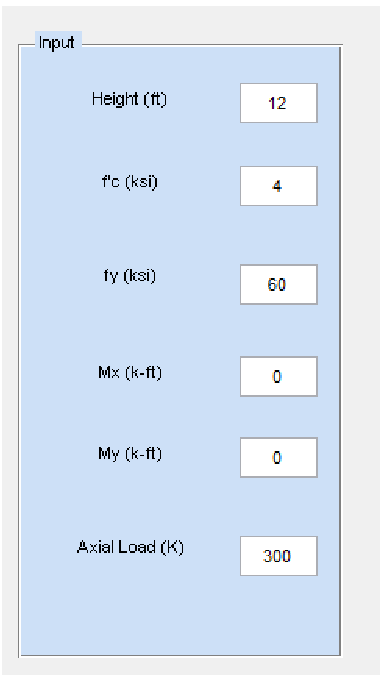
3→ is the yield strength of steel in ksi,

4→ is Mx = the moment applied on column in x-direction in kip ft units,

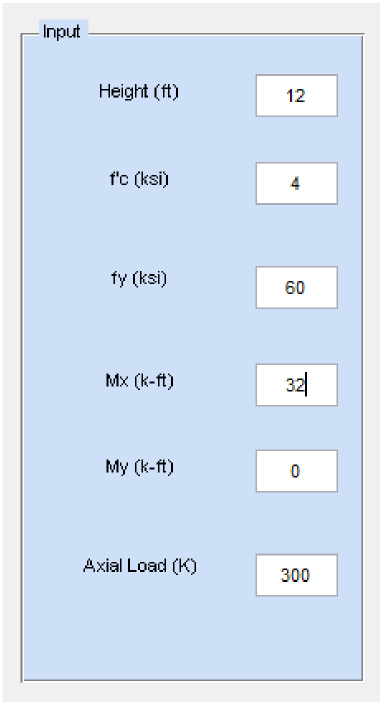
5→ is My = the moment applied on column in y-direction in kip ft units,

6→ is Axial load applied on column in kips

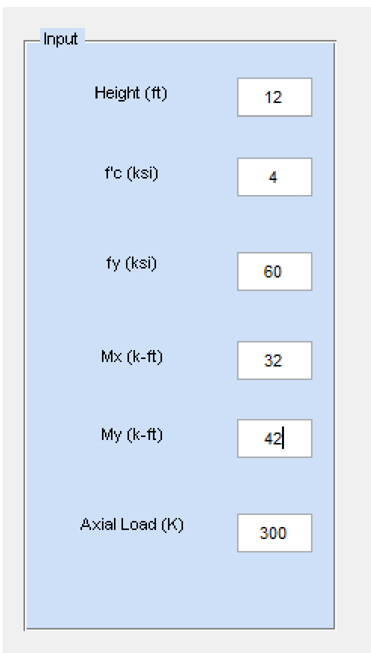
Users have option for axially loaded column, uni-axially loaded as well as bi-axially loaded column.



(Typical user input for axial column)

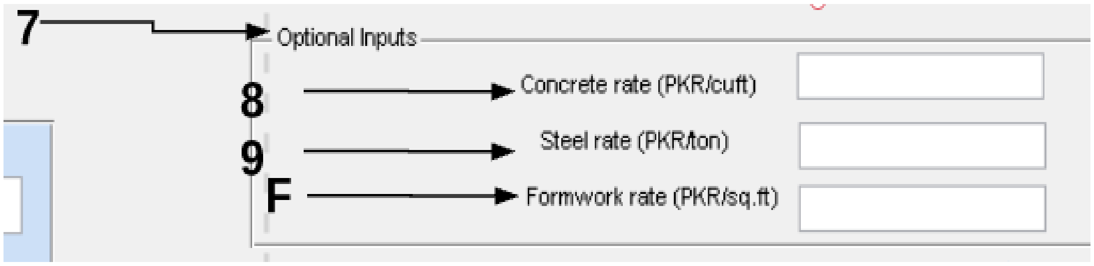


(User input for uni-axial column)



(User input for bi-axial column)

6.4.2 Input for market rates (Optional)



7→ is the optional inputs panel

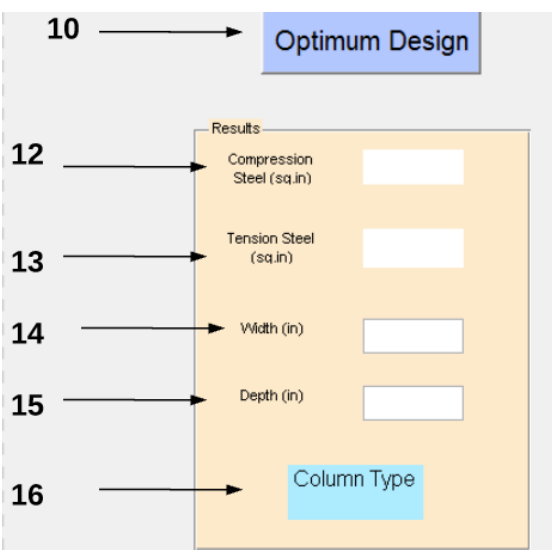
8→ is the concrete pouring rate in Pakistani rupee for one cubic ft

9→ is the steel rate in Pakistani rupee in terms of unit weight

F→ is the rate of formwork in Pakistani rupee per unit square ft

It is the optional input for user if he wants to input its own market rate in Pakistan. If user doesn't give the input for market rates then default **Military Engineering Services (M.E.S) rate of schedule 2014** will be taken as default input for market rates.

6.4.3 Results of Optimum Design



- When user have given the inputs for design then by pressing **Optimum Design** button user obtain optimized results. it is labeled as 10

12→ gives area of compression steel in square inch

13→ gives area of tension steel in square inch

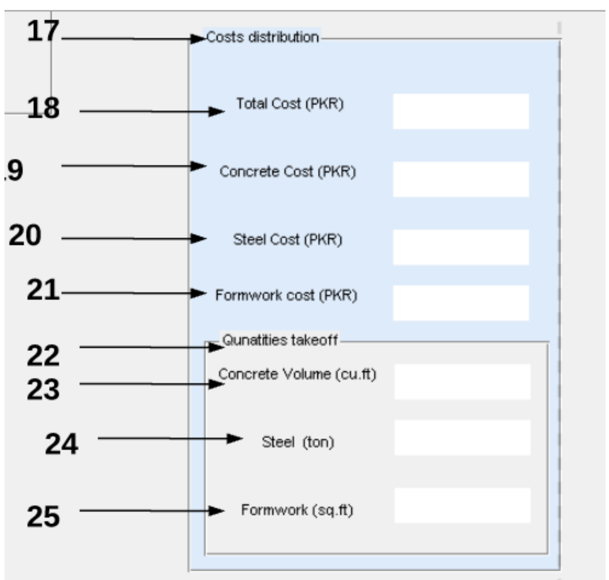
14→ gives width of column in inches

15→ gives depth of column in inches

16→ gives column type i.e. axially loaded, uni-axial and bi-axial

Since this software is user friendly so user can gives its **own cross-section dimension** in fore pressing optimize design button. The minimum value of width is 10 inch.

6.4.4 Results of Optimum Design (cost and Quantities Takeoff)



17→ is the cost distribution of frame among concrete,formwork and steel

18→ is the total cost of frame in Pakistani rupee

19→ is the cost of concrete in Pakistani rupee

20→ is the cost of steel in Pakistani rupee

21→ is the cost of formwork in Pakistani rupee

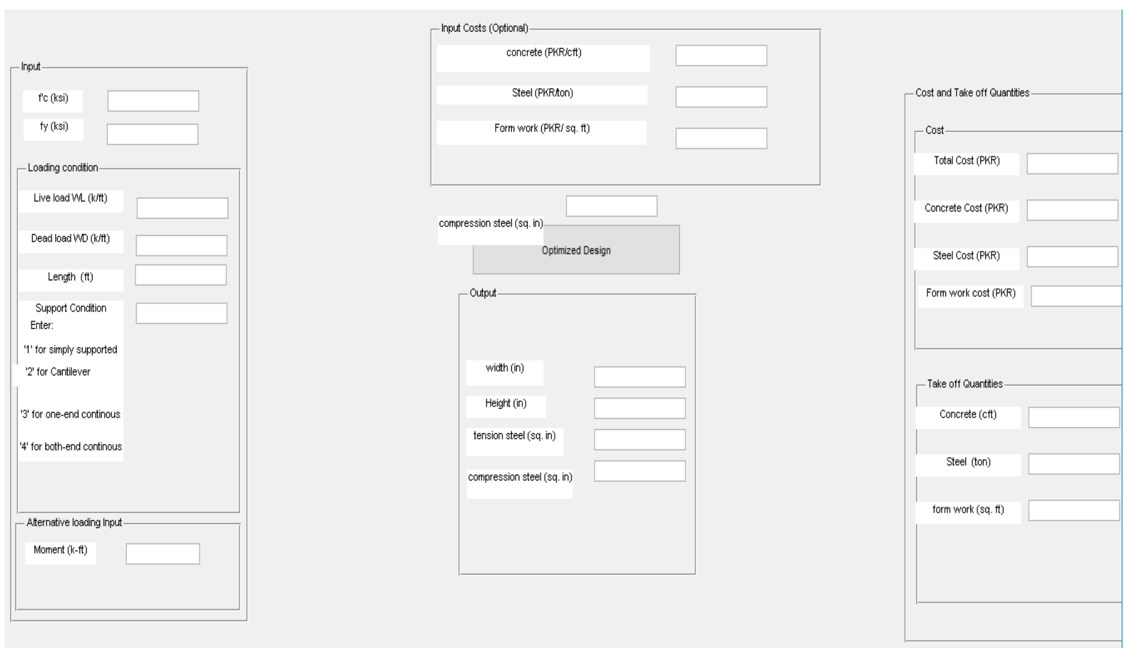
22→ is the quantities takeoff

23→ is the concrete volume of optimized frame in cubic ft.

24→ is the weight of optimized frame in ton.

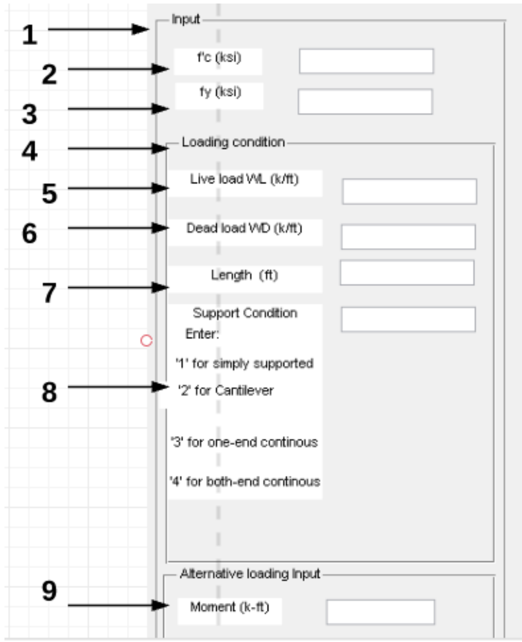
25→ is the formwork of optimized frame in square ft.

6.5 Using the SMAF Optimized Beam software



This is the graphical user interface (GUI) of Column module.

6.5.1 Input



1→ is the input panel for user inputs

2→ is the compressive strength of column in ksi,

3→ is the yield strength of steel in ksi,

4→ is the loading conditions input panel for user

5→ is the given live load in kip-ft.

6→ is the given dead load in kip-ft.

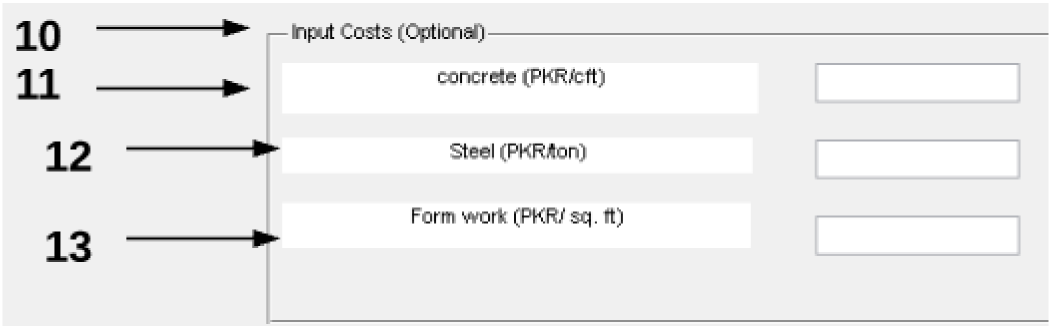
7→ is the length of beam in ft.

8→ is the support condition for user. Enter number in the box depending on the support condition.

- Enter 1 in the box for simply supported.
- Enter 2 in the box for cantilever.
- Enter 3 in the box for one-end continuous.
- Enter 4 in the box for both end continuous.

9→ is the alterative option for load in the form of moments in kp-ft.

6.5.2 Input for market rates (Optional)



10→ is the optional inputs panel

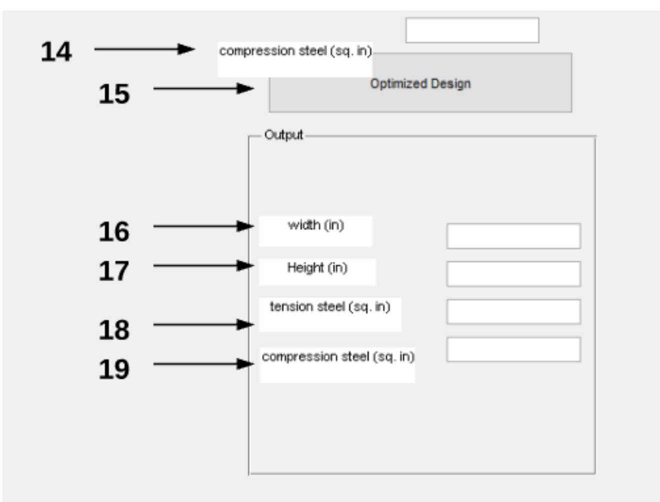
11→ is the concrete pouring rate in Pakistani rupee for one cubic ft

12→ is the steel rate in Pakistani rupee in terms of unit weight

13→ is the rate of formwork in Pakistani rupee per unit square ft

It is the optional input for user if he wants to input its own market rate in Pakistan. If user doesn't give the input for market rates then default **Military Engineering Services (M.E.S) rate of schedule 2014** will be taken as default input for market rates.

6.5.3 Results of Optimum Design



14→ is the compression steel in square inch

15→ is the optimized design button which will be pressed after giving inputs

16→ is the width of beam in inch after pressing optimized design button

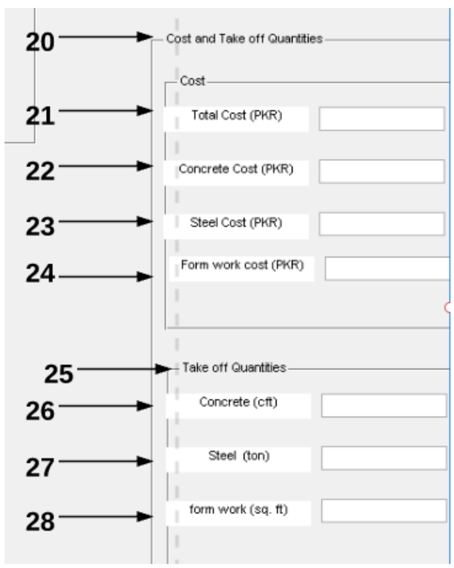
17→ is the height of beam in inch after pressing optimized design button

18→ is the area of tension steel in beam

19→ is the area of compression steel in beam

Since this software is user friendly so user can gives its **own cross-section dimension** in dialog box before pressing optimize design button.

6.5.4 Results of Optimum Design (cost and Quantities Takeoff)



20→ is the cost distribution of frame among concrete,formwork and steel

21→ is the total cost of frame in Pakistani rupee

22→ is the cost of concrete in Pakistani rupee

23→ is the cost of steel in Pakistani rupee

24→ is the cost of formwork in Pakistani rupee

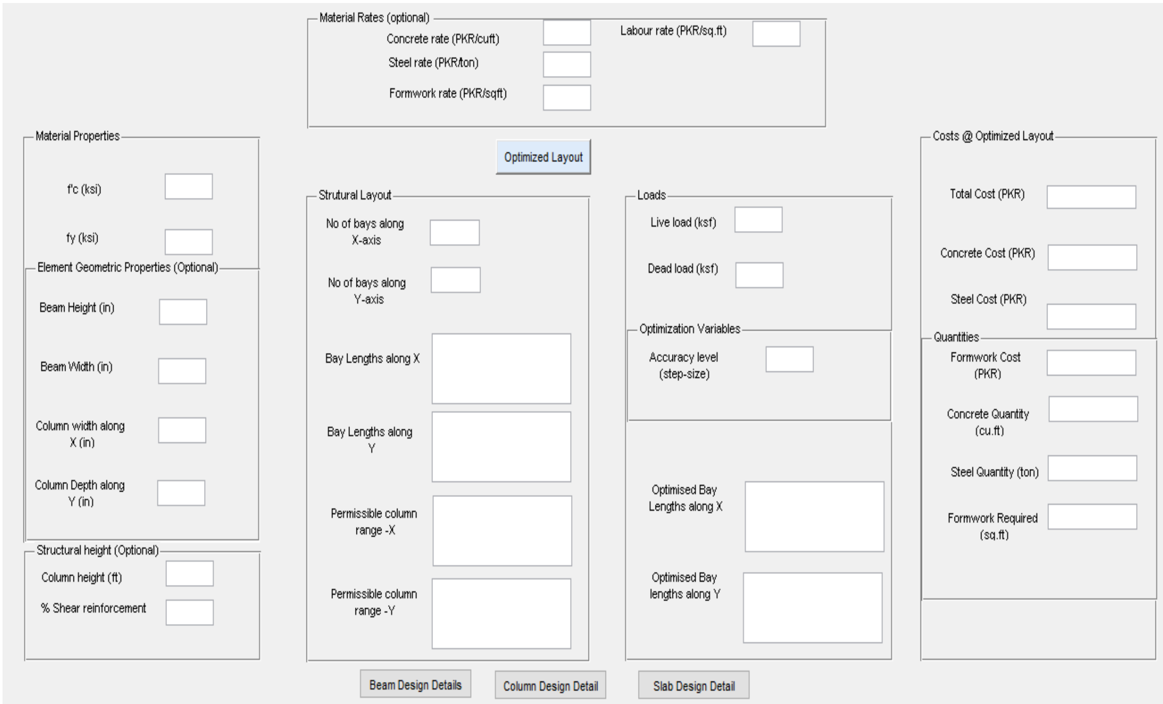
25→ is the quantities takeoff

26→ is the concrete volume of optimized frame in cubic ft.

27→ is the weight of optimized frame in ton.

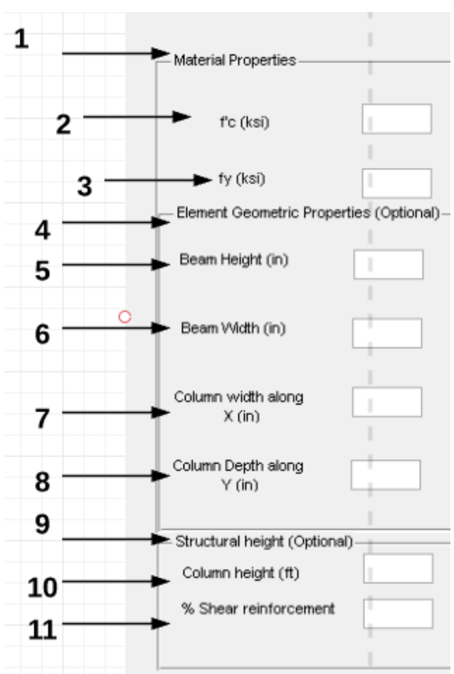
28→ is the formwork of optimized frame in square ft.

6.6 Using the SMAF Optimized Layout software



This is the GRAPHICAL USER INTERFACE (GUI) of Layout module.

6.6.1 Input



1→ is the material properties defined by user.

2→ is the compressive strength of column in ksi

3→ is the yield strength of steel in ksi

4→ is the geometric properties of elements of frame defined by user if he wants to input his own properties

5→ is beam height in inches

6→ is beam width in inches

7→ is column width in inches

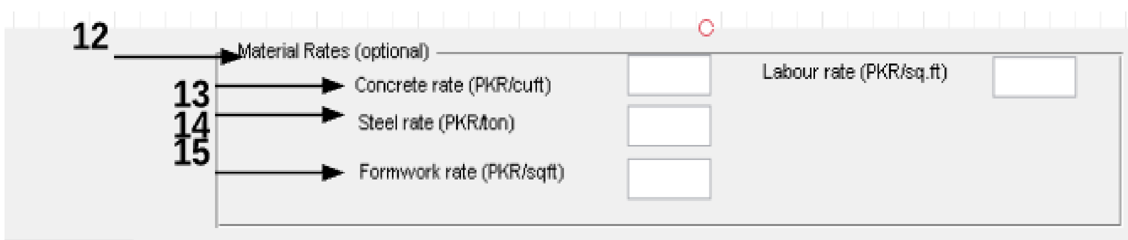
8→ is column depth in inches

9→ is structural height (optional tool)

10→ is column height in feet

11→ is % shear reinforcement

6.6.2 **Input for market rates (Optional)**



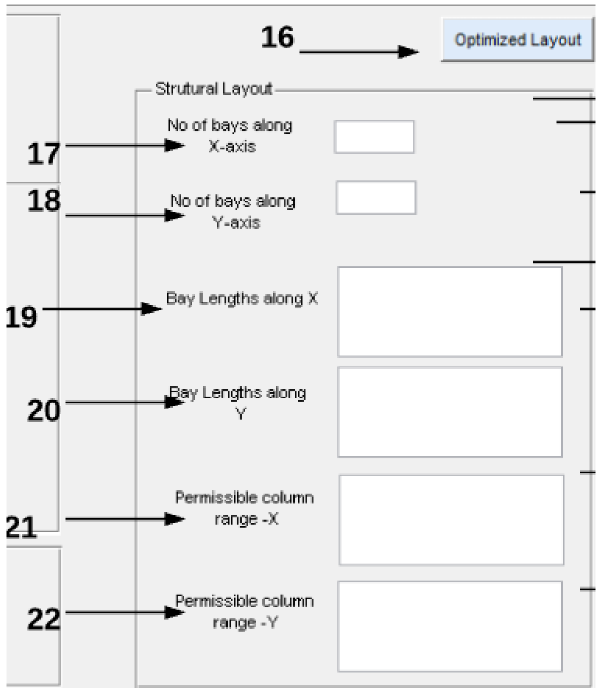
12→ is the optional inputs panel

13→ is the concrete pouring rate in Pakistani rupee for one cubic ft

14→ is the steel rate in Pakistani rupee in terms of unit weight

15→ is the rate of formwork in Pakistani rupee per unit square ft

It is the optional input for user if he wants to input its own market rate in Pakistan. If user doesn't give the input for market rates then default **Military Engineering Services (M.E.S) rate of schedule 2014** will be taken as default input for market rates.



16→ is optimized layout button (this is the button to get optimized layout of the frame)

17→ is no. of bays along X-axis

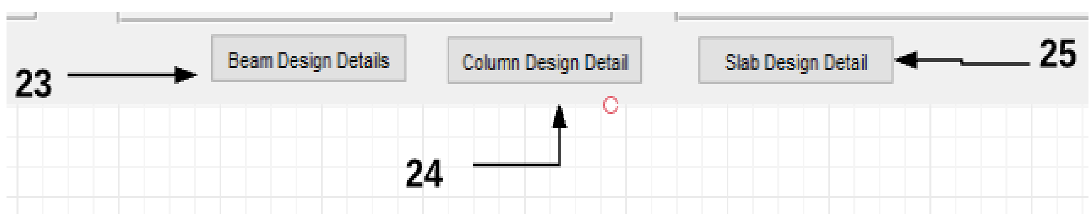
18→ is no. of bays along Y-axis

19→ is bay length along X-axis

20→ is bay length along Y-axis

21→ is permissible column range in X-direction (ft)

22→ is permissible column range in Y-direction (ft)

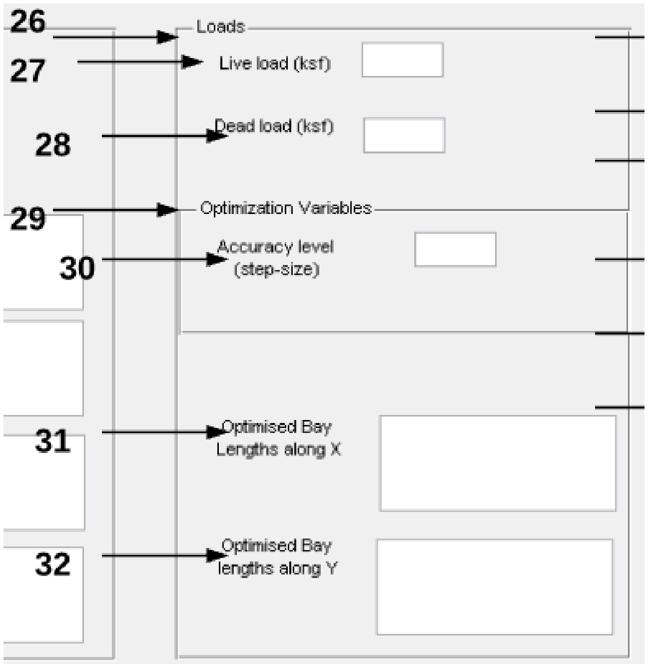


23→ gives beam design detail

24→ gives column design detail

25→ gives slab design detail

6.6.3 **Assigned Load and optimized layout**



26→ here you put loads (ksf)

27→ Enter Live loads (ksf)

28→ Enter Dead load (ksf)

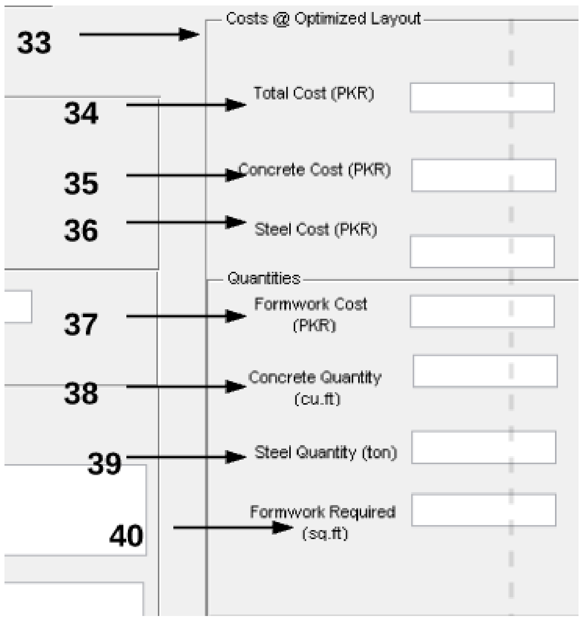
29→ optimization variable tool box

30→ Accuracy level (how many iterations)

31→ Optimized bay length along X-axis

32 → Optimized bay length along Y-axis

6.6.4 **Results of Optimum Design (cost and Quantities Takeoff)**



33→ Cost at optimized layout

34→ gives total cost (PKR)

35→ gives concrete cost (PKR)

36→gives steel cost (PKR)

37→ gives formwork cost (PKR)

38→ Amount of concrete required in cft

39→ Amount of steel required in ton

40→ Amount of formwork required in sq.ft

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