INVESTIGATING THE EFFECT OF COLUMN ORIENTATIONS ON MINIMUM WEIGHT DESIGN OF STEEL FRAMES

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ABSTRACT

INVESTIGATING THE EFFECT OF COLUMN ORIENTATIONS ON MINIMUM WEIGHT DESIGN OF STEEL FRAMES

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Steel has become widespread and now it can be accepted as the candidate of being main material for the structural systems with its excellent properties. Its high quality, durability, stability, low maintenance costs and opportunity of fast construction are the advantages of steel. The correct use of the material is important for steel's bright prospects. The need for weight optimization becomes important at this point. Available sources are used economically through optimization. Optimization brings material savings and at last economy. Optimization can be achieved with different ways. This thesis investigates the effect of the appropriate choice of column orientation on minimum weight design of steel frames. Evolution strategies (ESs) method, which is one of the three mainstreams of evolutionary algorithms, is used as the optimizer in this study to deal with the current problem of interest. A new evolution strategy (ES) algorithm is proposed, where design variables are considered simultaneously as cross-sectional dimensions (size variables) and orientation of column members (orientation variables). The resulting algorithm is computerized in a design optimization software called OFES. This software has many capabilities addressing to issues encountered in practical applications, such as producing designs according to TS-648 and ASD-AISC design provisions. The effect of column orientations is numerically studied using six examples with practical design considerations. In these examples, first steel structures are sized for minimum weight considering the size variables only, where orientations of the column members are initially assigned and kept constant during optimization process. Next, the weight optimum design of structures are implemented using both size and orientation design variables. It is shown that the inclusion of column orientations produces designs which are generally 4 to 8 % lesser in weight than the cases where only size variables are employed.

Keywords: Optimization, Structural optimization, Evolution algorithms, Evolution strategies, Structural design, Steel frames, Optimal choice of column orientations.

ÖZ

KOLON DOĞRULTULARI SEÇİMİNİN MİNİMUM AĞIRLIKLI ÇELİK ÇERÇEVE YAPI TASARIMINA ETKİSİNİN İNCELENMESİ

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Mükemmel özellikleri ile yapısal sistemlerin birinci öncelikli malzemesi olmaya aday olan çelik giderek yaygınlaşmaktadır. Çeliğin yüksek kalitesi, dayanıklılığı, stabilitesi, düşük bakım masrafları ve hızlı inşası avantajlı yönleridir. Malzemenin doğru kullanımı çeliğin parlak geleceği için önemlidir. Ağırlık optimizasyonu bu noktada önem kazanmaktadır. Mevcut kaynaklar optimizasyonun devreye girmesi ile en ekonomik şekilde kullanılmaktadır. Optimizasyon malzeme tasarrufunu ve ekonomiyi sağlamaktadır. Optimizasyon farklı yollarla gerçekleştirilebilir. Bu tez, kolonlarda uygun doğrultu seçiminin minimum ağırlıklı çelik yapı tasarımına etkisini incelemektedir. Evrimsel algoritmanın üç ana dalından biri olan evrimsel stratejiler (ESs) metodu bu çalışmadaki problemlerin çözümünde optimizasyon aracı olarak kullanılmıştır. Tasarım değişkeni olarak aynı anda kesit alan boyutlarını (boyut değişkeni) ve kolon elemanlarının doğrultularını (doğrultu değişkeni) dikkate alan yeni bir evrimsel strateji (ES) algoritması ileri sürülmüştür. Ortaya çıkan algoritma, tasarım optimizasyonunun yapıldığı yazılım programı OFES'te kullanılmıştır. Bu program, pratikte yer alan uygulamalarda TS–648 ve ASD-AISC standartlarına göre çözümler üretebilmektedir. Kolon oryantasyonunun etkisi, altı örnekle pratik tasarım esasları doğrultusunda sayısal olarak çalışılmıştır. Bu örneklerde ilk olarak,

kolon doğrultuları önceden belirlenmiş ve sabit olarak bırakılmış çelik yapılar, sadece boyut değişkenleri dikkate alınarak boyutlandırılmıştır. Daha sonra, bu yapıların optimum ağırlık tasarımları hem boyut hem de doğrultu tasarım değişkenlerinin kullanılmasıyla gerçekleştirilmiştir. Kolon doğrultularının tasarım değişkenlerine eklenmesi, sadece boyut değişkenlerinin kullanıldığı durumlara oranla genellikle % 4 ile 8 arasında daha hafif tasarımlar elde edilmesini sağlamaktadır.

Anahtar Kelimeler: Optimizasyon, Yapı optimizasyonu, Evrimsel algoritmalar, Evrimsel stratejiler, Yapı tasarımı, Çelik çerçeveler, Kolon doğrultularının en uygun seçimi.

To My Parents and Grandmother,

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I'm grateful to my mother Beyhan Kızılkan and my father Veli Kızılkan for giving their love, patience and support during my endless school life and it was my chance that each time despair knocks the door they was there to give their helping hand. It is beyond description for the author to express her intense feelings to her parents and she is praying for a lifetime support of them.

I would embrace my grandmother Hadiye Eper with pure love and respect for her warm and compassionate receiving with open arms; she was the one who entertains a kindly feeling for me.

I should mention that it would be a proud of mine to try to pay my gratitude to my aunt, my uncle, my cousins, my aunt in law and my uncle in law, who is far away from us, and to İzmir and to all the ones who lives in İzmir.

I would have been incomplete, if I didn't have my dearest friends with whom life blossoms and is a pleasure to see them after awakening from the ever darkest nights.

The author presents her respects to PROKON Engineering and to her colleagues who have great helps and supports to her, while she was both studying and working. It was my chance to walk with them all through the same way.

My endless thanks are for my school and solution partners, they were always there with their cheerful personalities all through my limited time in school, and the author is seeking for a lifetime friendship.

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A_i	Length of the Steel Section
$\boldsymbol{\alpha}$	Rotation Angle (Correlation Coefficient)
b	The Best Design
b_f	Flange Width of the Beam or Column
C_m	Moment Coefficient
C_v	Web Shear
d_c	Depth of the Column
E	Elasticity Modulus of Steel
F_y	The Yield Stress of Material
F_a	The Allowable Axial Stress
F_b	The Allowable Bending Stress
f_a	Computed Axial Stress either in Compression or in Tension
f_b	Computed Normal Stress in Bending.
f_{bx}	Computed Normal Stress about Major Bending Direction
F_{bx}	The Allowable Major Bending Stress
f_{by}	Computed Normal Stress about Minor Bending Direction
F_{by}	The Allowable Minor Bending Stress
F'_{ex}	Euler Stress about Principal Axis of the Member
F'_{ey}	Euler Stress about Principal Axis of the Member
f_v	Computed Shear Stress
F_v	Allowable Shear Stress
H	Total Height of the Frame Building
I	Size Design Variables
i_b	Radius of Gyration
K	Effective Length
L	Element Buckling Length
N_m	Number of Members

N_d	Sizing Groups
N_o	Number of Orientation Groups for Column Members
O	Orientation Design Variables
p_s	Relative Frequency
P(0)	Initial Population
$P(t+1)$	Parent Population of the Next Generation
p	Mutation Probability
r	Minimum Radius of Gyration
s_b	Unbraced Member Lengths
$s(\phi)$	Adaptive Strategy Parameters
s	Arbitrary Component of an Individual
t_f	Flange Width of the Column
$\mathbf{x}$	Design Variable Vector
W	Weight of the Frame
$\mathbf{z}$	N-dimensional Random Vector.
α	Penalty Coefficient
μ	Parent
λ	Offspring
σ	Standard Deviation
ξ	Mean
κ_i	Distributed Integer Random Number
ρ_i	Unit Weight of the Steel Section
Ψ	Set of Geometric Distribution Parameters
ϕ	Constrained Objective Function Value
γ	Learning Rate
τ	Learning Rate

CHAPTER 1

INTRODUCTION

Developments in computer technology, advances in material quality and the idea of seeking for the best solution accelerated the studies on economical structural systems that can be analyzed in short durations. In the last decade developments in architecture and increasing demands for high rise buildings resulted in systematic design of steel structures.

Steel plays an important role in this development process. It not only exhibits certain advantages over other materials in terms of its mechanical characteristics, such as high strength and ductility, but also offers an opportunity for assembling different structural frame systems for massive parts.

The idea of using steel is also related with gravity loads. As the building becomes higher the columns from top to bottom are subjected to greater loads. Steel yields both aesthetic and economical solutions to such structures. It is possible to use different sections of columns from bottom to top without compromising aesthetics.

It is important to use optimization techniques in steel, whose superior material properties are implied. A structural system that is not only strong enough to satisfy the limitations but also light enough to minimize the usage of natural sources can be satisfied with the correct choice of members. A light weighted structure will be in benefit of environmental factors and also will help the minimization of the earthquake forces. Slender members that satisfy the limitations will help the aesthetic concerns.

Mathematical programming techniques and optimality criteria have long been used in structural optimization problems. The design variables were assumed to be continuous in these derivation-based methods. At the end of the optimization, the results were chosen from continuous design sets. Usually the results were not relevant to the practical fabrication requirements.

The studies continued to overcome these drawbacks of the existing methods, and accordingly new techniques have emerged. Metaheuristic search algorithms, which use nature as a source of inspiration to develop numerical solution algorithms, have five branches. Simulated annealing, evolutionary algorithms, tabu search, harmony search, swarm-based optimization are the techniques that belong to the metaheuristic search algorithms. These algorithms do not require any gradient information of the objective function and constraints and the transition rules are not deterministic any more, instead probabilistic transition rules are used [1]. There is another attractive feature of metaheuristic search algorithms. In addition to continuous variables, these methods can also deal with discrete design variables, which in fact give an opportunity to the designer to select members from a list of ready sections.

1.1 OBJECTIVES

Columns and beams are the main members of steel structures. The relevant orientation of the elements plays an important role in the weight of the system. Each of the member groups should be placed in right position so that their strong axis can resist on the exerted forces. When designing space steel structures, beams are placed such that the major bending axis coincides with the strong axis of the beam, whereas columns are oriented in any direction depending on the intuition of the designer.

The column orientations can be predicted according to some design heuristics. If the plan of the structure is square or almost square the system needs to have the same rigidity in both directions. This can be satisfied with column orientations. The number of columns whose strong axes are in the same direction with x axis of the structure should be equal to the one's that are in y direction. By this way the building's resistance to the lateral loads will be similar in both directions. In case of a rectangular building, columns are placed with their strong axis perpendicular to short side in order to increase resistance of the building against bending along short side.

The optimum design process of the steel frames in the literature is only based on sizing of structural members in which orientations of the columns are determined initially and kept constant during optimum design process. The objective of the thesis is to investigate the effect of the choice of column orientation on minimum weight design of steel frames. Evolution strategy method, which is one of the three mainstreams of Evolutionary Algorithms, is used as the optimizer in the study to deal with the current problem of interest. In the study, a new ES algorithm is proposed, where design variables are considered simultaneously as cross-sectional dimensions (size variables) and orientation of column members (orientation variables.)

1.2 SCOPE

The rest of the thesis is organized as follows:

In Chapter 2, literature survey is carried out. Preceding studies on the use of evolution strategies technique in structural optimization applications are briefly summarized. It is emphasized that the literature lacks studies where orientation of the columns are used as design variables in the optimum design process of steel frames.

In Chapter 3, evolution strategies (ESs) method is introduced thoroughly. The development of the method and its enhancements over time are mentioned. It is noted that the method was originally developed as a continuous variable optimization technique, and later it was reformulated in the literature to deal with discrete variables also. Although not used effectively in the thesis, continuous ESs and their variants are overviewed first to describe the fundamentals of the technique. Next, various reformulations of discrete ESs proposed in the literature are overviewed.

In Chapter 4, the problem formulation regarding the weight optimum design of space steel structures are presented. In this chapter, the stress, stability and displacement imposed according to Allowable Stress Design- American Institute of Civil Engineers (ASD-AISC) are formulated. In addition, geometric constraints that imposed considering practical requirements of the design process are also formulated.

Chapter 5 describes about the ES algorithm developed for optimum design of steel frames where size and optimization variables are implemented simultaneously to minimize the structural weight of such systems. The resulting is computerized in a software called OFES. The capabilities and practical features of the software are also discussed here.

Numerical examples are presented in Chapter 6. Six design examples are studied in all to scrutinize the effect of column orientation on minimum weight design of steel structures. For each design example, full design data including loadings, element definitions and member groupings is provided. Each example is first designed for minimum weight considering size variables only, where orientations of the columns are assigned initially and kept unvarying during the optimum design process. Next, the example is reworked in which a minimum weight design is sought for the same system by taking both size and orientation variables as the design variables in the process. Optimum designs are reported in each case in terms of the discrete sections attained for member groups and orientations of the column members in optimum design model. A comparison is carried out between these cases to quantify the effect of the choice of column orientation.

Finally Chapter 7 concludes the thesis and summarizes some important results of the study.

CHAPTER 2

LITERATURE SURVEY / BACKGROUND

2.1 REVIEW OF THE LITERATURE

Metaheuristic techniques imitate the paradigm of natural evolution observed in biological organism to improve a set of designs using the evolutionary principles. Evolutionary algorithms belong to the group of metaheuristic techniques in which evolution strategies is placed. Evolution strategy is employed as the tool for optimization in this study. In the following, major studies in the literature that employ ESs method in structural optimization applications are briefly overviewed.

Papadrakakis *et al.* [2] studied structural optimization using evolution strategies and neural networks. It is mentioned that a gradient based optimization has the drawback of performing a great deal of sensitivity analysis. On the other hand evolution strategies do not require any sensitivity analysis. They have an advantage of robustness and better global behavior and disadvantage of the method is slow rate of convergence towards the global optimum. In the light of previous studies on artificial neural networks (ANN) the use of ANN in ESs in sizing and shape problems is investigated and drawbacks of ESs method are tried to be eliminated. Structural problems are solved in the scope of the study to see the advantages of the new combinatory method. The selection of the training set is also investigated. Randomly chosen combinations of input data using a Gaussian distribution around the midpoints of the design space is accepted as the best training set selection scheme. The study concluded that, combination of ANN and ES is quite useful and advantageous for optimization. The time consuming parts of optimization process are handled in relatively short time intervals.

Multi objective discrete optimization of laminated structures was performed by Spallino and Rizzo[3]. A multi-objective design problem refers to a case where more than one objective function is defined. Spallino and Rizzo started to work in line with the work of Pareto [4] who has studied multi objective optimization first. In the study, multi-objective design problem for laminated composite structures is carried out using two different examples. A rectangular laminated plate is studied with design objectives of critical buckling load, critical temperature rise and failure load maximization. A laminated cantilever plate is the second problem of the study where the minimization of tip displacement and rotation and the minimization of the first natural frequency are taken as the design objectives. The problems have different loading and layouts but both have discrete design variables. The problems are solved with a method that is based on ES and game theory (bargaining method). The study came up with a result that the combination of ES with game theory is useful for the multi objective optimization [3].

Gutkowski, Iwanow and Bauer [5] concentrated on design of structural systems with minimum weight. The idea behind their study is to design the structure with controlled mutation. In this application, mutations are controlled by stresses whereas they are controlled by state variables in the others. Largest and smallest stresses are randomly verified in the optimized problems for the control step of the algorithm [5]. The study concluded that this method marks out for a brilliant future and studies can be continued for this method.

Lagaros *et al.* [6] is concerned with the improvement of the performance of the optimization procedure by modifying evolutionary algorithm. They discuss that genetic algorithms and evolution strategies have several advantages compared to other approaches. However, the major drawback of these approaches lies in their time consuming processes. Mathematical programming method is integrated to both genetic algorithms and evolution strategies for the purpose of forming hybrid methodologies to eliminate the aforementioned drawbacks of these techniques. Sequential quadratic programming (SQP), which is regarded as the most robust mathematical programming method, is included to the algorithm to increase the efficiency of these techniques. Numerical examples are performed to evince the efficiency gained with the hybrid methods.

Ebenau *et al* [7] developed an advanced evolution strategy algorithm with an adaptive penalty function for mixed-discrete structural optimization. In this work it is clearly stated that a small reduction of weight may also lead to a considerable decrease of the costs of manufacturing, transport and assembly. It is also stated that low weight of profiles means that slender members are in use and this brings about the problem of buckling. The need of a program that investigates not only buckling problems but also stability effects is exposed. In this study, these issues are taken care of by adding an adaptive penalty function to the (μ+1)-evolution strategy. The numerical effort is reduced with the help of (μ+1)-evolution strategy in the study. A mutation procedure was added to the computational steps which gives the algorithm an opportunity to reach the global optimum. The study is concluded that most convenient penalty function is the one which depends on both the constraint value and actual rate of feasible individuals in the current population. It is noted that optimization with mixed-discrete variables can be solved with the variant of (μ+1)-evolution strategy. The adaptation to the strategy is the utilized mutation operator and the combination with a penalty function that works robust and efficient [7].

In Lagaros *et al.* [8] the performance improvement of the evolution strategies in the structural optimization is addressed. This improvement is investigated in conjunction with especially large scale structures. In the study neural network (NN) strategy is used to replace computationally expensive finite element analyses required by ESs with inexpensive and acceptable approximation of the exact analysis. The algorithms are implemented on two computing platforms; sequential and parallel computing environments. Different adaptive NN training schemes are examined to find the best performance. Both large and small starting training sets are chosen. It is observed that small initial training sets are better than the large ones from performance standpoint. The idea behind the strategy of adaptively creating the NN training set is providing an NN configuration gradually with prediction capabilities for the regions of the overall design space that are actually visited by the ES procedure [8].

In study of Rajasekaran *et al.* [9] space structures are optimized for minimum weight with functional networks. It is stated that classical optimization methods are not appropriate for large scale structures, because of time consuming sensitivity analysis. On the other hand, probabilistic search methods, (such as ESs), are not gradient based methods so that no sensitivity analysis is required for their implementation. Formex algebra of the Formian software [10] is chosen to generate the geometry of large scale space structure. Encouraging results are obtained for several design examples considered in the study and it is concluded that the method proposed is a very efficient method.

In another study by Baumann and Kost [11][10] , topology optimization of discrete structures such as trusses is studied with ESs technique. It is stated that the most common approach for topology optimization is ground structure method. With this method, unnecessary elements of highly connected initial structures are eliminated until the objective function is reached.

CHAPTER 3

EVOLUTION STRATEGY METHOD

3.1 SEARCH TECHNIQUES

As shown in Figure 3.1 search techniques of optimization techniques can coarsely be classified into three main groups as enumerative techniques, calculus based techniques and global optimization techniques.

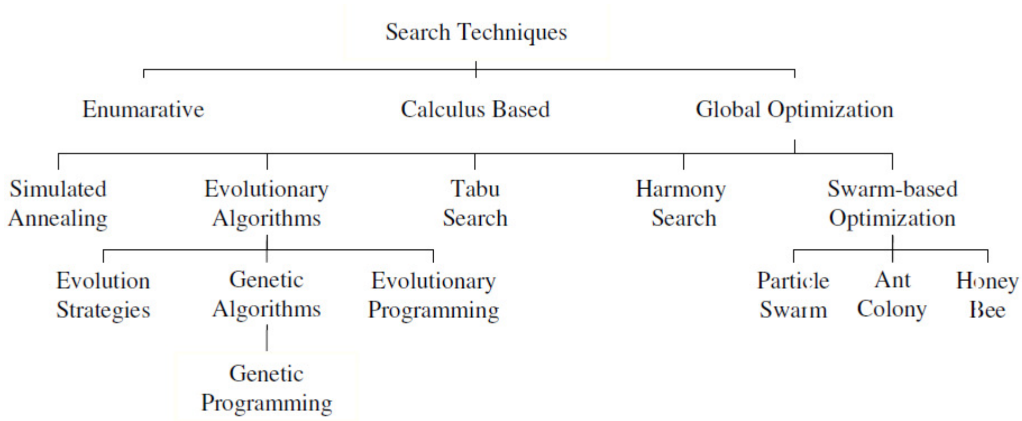


Figure 3.1 Categorization of search techniques

Enumarative techniques, in principle, search every possible point in design space or domain one point at a time. They can be simple to implement but the number of possible points may be too large for direct search. *Calculus based techniques* use the gradient values to estimate the location of nearby optimum. These techniques are known as *Hill Climbing* techniques because they estimate where the maximum lies, move to that point, make a new estimate, and make a new move and repeat this process until they reach the top of the hill.

Global optimization techniques have received increased attention, primarily because of their lack of dependence on gradient information, a more robust approach to handle discrete and integer design variables, and for an enhanced ability to locate the globally optimal solution; particularly in discrete optimization problems.

In the past, conventional optimization techniques (optimality criteria and mathematical programming methods) overwhelmingly controlled the early applications in the area of structural optimization. The majority of conventional techniques work on the basis of derivation of objective function and constraints with respect to design variables, that is to say, they accomplish a gradient-based search. However, this situation highly hampers their applicability to complex structural optimization problems. There are several reasons for this. Firstly, for a proper implementation of the gradient-based search, they necessarily require a continuous design space, where the design variables can assume any value between the specified bounds. Secondly, even if the requirement for continuity of the design space is satisfied, the gradient-based search followed by these techniques guides the process towards a point which is usually the local optimum closest to the starting solution. Considering that the design space incorporates many local optima, it is a very difficult task to reach the global optimum, avoiding all those local optima. Accordingly, their success is intimately dependent on the choice of a good starting solution, which is in most cases unknown at the start. Finally, they are not well suited to discrete variable optimization, where the design variables are to be selected from an already available list, rather that being continuous, e.g., integer and 0-1 optimizations can easily be interpreted as two special cases of the general discrete variable optimization approach. In particular, in civil engineering structures; steel structures representing a very large and important group in this respect, one has to choose the structural members from commercially available profiles in the markets, and the number of bolts used for connections match an integer number, etc. A customary approach followed by conventional techniques to handle a discrete variable problem is first to solve the problem using continuous variables, and then to round up the solution to the nearest existing discrete values. But, this approach may easily lead to non-optimum or infeasible solutions. Therefore, a computationally complex problem of this nature calls for an efficient and reliable optimization method.

Recently, a number of global optimization techniques have emerged to be promising strategies, showing certain superiorities over conventional techniques in these aspects, together with their potential applicability for a wide range of diverse problem areas. The underlying concepts of these techniques and thus their algorithmic models have been constituted by establishing correspondences between the optimization task and events occurring in nature, i.e., nature is used as a source of inspiration. This feature in turn brings about a solution methodology which completely rejects a gradient-based search so as to reduce the possibility of getting stuck in a local optimum. The most popular techniques in this category are evolutionary algorithms (EAs), simulated annealing (SA), tabu search, harmony search, and swarm-based optimization techniques as depicted in Figure 3.1.

3.2 GLOBAL OPTIMIZATION

Evolutionary algorithms refer to a group of techniques, which imitate the paradigm of natural evolution observed in biological organism to improve a set of designs using the evolutionary principles. Genetic algorithms, evolution strategies and evolutionary programming are regarded as the three mainstreams of evolutionary algorithms. Genetic algorithms were first introduced by Holland [12]; early studies in evolution strategies were pioneered by Rechenberg [13],[14] and Schwefel [15]; and evolutionary programming was first put forward by Fogel [16]. The procedure used in any evolutionary algorithm technique requires a stochastic and iterative process, which endeavors to improve a population of designs (individuals) over a selected number of generations.

The optimization task in the simulated annealing (SA) is achieved by following another heuristic concept extending to the annealing process of physical systems in thermodynamics. In this process, a physical system initially at a high energy state is cooled down to reach the lowest energy state. The idea is that this process can be mimicked to handle optimization problems is accomplished by Kirkpatrick *et al.* [17], establishing a direct analogy between minimizing the energy level of a physical system and lowering the objective function

Particle swarm optimization technique was developed by Eberhart and Kennedy [18]. The technique is based on the idea of animal flocking. Each solution in the swarm is called as particle and they are compared to the best solutions that are reached before according to their fitness values. Optimum particles are determined and all of the particles follow the current optimum particles to find the best. Initially, particles compose a group and in each iteration, particles are placed with the optimum one until the best solution is reached and limitations are satisfied [19]. The memory that each individual has and the knowledge that the swarm gained constitutes the behavior of the system. The swarm is represented with a number of particles and these particles are initialized randomly in the search space of an objective function[1].

Tabu search was developed by Glover [20]. The way the technique is implemented shows a strong similarity with the meaning of the word "*tabu",* which implies a social or cultural restriction [21]. The main idea behind tabu search is to protect the search from local optima. For this purpose a tabu list, a short term memory, is prepared which includes recently visited solutions or candidate solutions which the search is prohibited to be transmitted to. Another type of memory, long term memory is also used to direct the solutions to a predefined point. Selection, reproduction, mutation, searching from tabu list satisfying aspiration criterion and termination are the main steps of the method[22].

Idea of ant foraging by pheromone communication to form paths was the inspiration for *ant colony optimization* developed by Dorigo [23] that studies the method first. Ants use several paths to find food and they secrete pheromone behind to designate the path. This secretion looses its intensity with time. Intensity of each path helps the other ants to choose the shortest way that is intensive than the others. This phenomenon is the starting point of ant colony optimization. The main steps of this method are initialization of pheromones, selection probabilities, constructing a colony of ants, evaluation of the colony, global pheromone update, pheromone scaling and termination [1].

Harmony search developed by Lee and Geem [24] is mimicked from musical performance process that takes place when a musician searches for a better state of harmony. Jazz improvization seeks musically pleasing harmony similar to the optimum design process which seeks to find the optimum solution. The pitch of each musical instrument determines the aesthetic quality, just as the objective function value is determined by the set of values assigned to each decision variable. In the process of musical production a musician selects and brings together number of different notes from the whole notes and then plays these with a musical instrument to find out whether it gives a pleasing harmony. The musician then tunes some of these notes to achieve a better harmony. Likewise, a candidate solution is generated in the optimum design process by modifying some of the decision variables. This algorithm is based on four main steps, which are initializing a harmony memory, improvising a new harmony, exchanging the better harmony with the previous one and termination [25].

3.3 EVOLUTION STRATEGIES

Evolution Strategies are a subclass of Evolutionary Algorithms which were developed by Rechenberg and Schwefel at the Technical University of Berlin in 1964 for continuous design. The technique has been reformulated later by various researchers in the literature to deal with discrete optimization problems [26], [27] and [28]. It is an optimization technique based on adaptation and evolution.

As being a member of evolutionary algorithm class, evolution strategies are also inspired from evolutionary biology which incorporates reproduction, mutation, recombination, natural selection and survival of the fittest genetic operators for the implementation. First, a population of designs (solutions) is generated randomly in design space. Each design is referred to as a member or an individual and represents a complete solution to a problem at hand. In *selection*, the members that are more fit than others in the population survive and they become the parents for the next generation. The new members (offspring) are reproduced by means of recombination and mutation operators. *Recombination* is applied between parent and individuals by the exchange of genetic information between them to produce new members. *Mutation* is applied on new members to modify their genetic structures. The offspring on average will be better than its parents and they will compete with each other, and also with the older members (parents) to take place in the next generation.

Evolution strategies use natural problem-dependent representations, and mutation and selection as the primary search operators. These operators are applied in a loop called a generation which is continued until a *termination criterion* is met.

3.4 EVOLUTION STRATEGIES WITH CONTINUOUS DESIGN VARIABLES

An experimental optimization technique was raised in 1964 by Rechenberg and Schwefel which was formerly used to solve the problem of driving a flexible pipe bending or changeable nozzle contour into a shape with minimal loss of energy [29]. The problem was solved by an early version of evolution strategies referred to (1+1)-ES, which is based on one parent and one offspring per generation. If the offspring is better than the parent, offspring becomes the new parent of the next generation otherwise the parent survives.

As the studies improved, an adjustment rule was developed called 1/5 success rule for an adaptive implementation of mutation operator. By the use of the rule, the ratio of successful mutations to all mutations is adjusted to be equal to 1/5. It is the idea of having only 1 successful child in every 5 mutation, Equation (3.1). If this ratio is greater than 1/5, the step size (σ) is increased to make a wider search of the space, and if the ratio is less than 1/5 then it is decreased to concentrate the search more around the current solution [30].

$$\sigma = \begin{cases} \sigma / c & if \quad p_s > 1/5, \\ \sigma \cdot c & if \quad p_s < 1/5, \\ \sigma & if \quad p_s = 1/5 \end{cases} \tag{3.1}$$

In the above equation σ, the standard deviation, is either divided to the parameter c or multiplied with c according to the relative frequency, p_s's, of the successful mutations measured over a number of trials, ratio. The parameter c is any value between $0.817 \leq c \leq 1$.

The very first version of evolution strategies was used for continuous design variables. The most important step is the mutation that adaptively adjusts itself online during the search. An outline can be given, for a two member algorithm, to clarify how the method works [30].

Algorithm (1+1)-ES;

BEGIN

set t=0;

Create an initial point $(x_1^t,...,x_n^t) \in \Re^n$;

REPEAT UNTIL (TERMINATION CONDITION is satisfied) DO

draw z_i from normal distr. for all $i \in \{1,...,n\}$ independently;

$y_i^t = x_i^t + z_i$ for all $i \in \{1,...,n\}$;

IF $(f(\bar{x}^t) \leq f(\bar{y}^t))$ THEN

$\bar{x}^{t+1} = \bar{x}^t$;

ELSE

$\bar{x}^{t+1} = \bar{y}^t$;

FI

set $t = t + 1$;

OD

END

This early variant of the evolution strategies was soon replaced by more advanced and improved multi-member versions of the technique, where the number of parents (*μ)* and the number of offspring (*λ*) employed at a generation are taken greater then one, i.e., μ>1 and λ>1. However before such state of the art variants of the technique, an intermediate algorithm referred to as (μ+1)-ES was developed. *Recombination* concept is first introduced with this version. In this version, one offspring is created at each generation; this design substitutes the worst parent individual in the population following mutation and fitness evaluation.

The modern variants of the technique are known as (μ+λ)-ES and (μ, λ)-ES. Basic principles of these strategies should be mentioned first. In *(μ+λ)* ES, μ parents create λ offspring by recombination and mutation and the μ best designs are selected as the parents of the next generation from both μ parents and λ offspring deterministically. In case of a *(μ, λ)*-ES, the parents are not included in the selection mechanism; instead selection is carried out by choosing the μ best individuals out of λ offspring in reference to the individuals' fitness scores. In this case $\lambda \geq \mu$ is required, otherwise $\lambda>1$ is sufficient [31].

An initial population of μ parent individuals is created randomly in order to initiate the process from a number of arbitrary search points in the design space. Each individual consists of a certain number of strategy parameters in addition to a problem-specific set of design variables, which are all represented by their actual numerical values. Each individual of the parent population is evaluated and assigned a fitness value according to the objective function considered. The next step is to recombine and mutate the parent population to generate the offspring. These offspring designs are also evaluated. The selection is carried out between parent and offspring designs to determine the parents of the next generation. In the following, the computational steps of the algorithm are discussed in more detail.

3.4.1 Basic Concepts in Evolution Strategies

3.4.1.1 Representation

For a general problem, an individual $\mathbf{I}$ may consist of up to three different components ($\mathbf{x}, \boldsymbol{\sigma}, \boldsymbol{\alpha}$) and can be defined as follows:

$$\mathbf{I} = (\mathbf{x}, \boldsymbol{\sigma}, \boldsymbol{\alpha}) \tag{3.2}$$

Here, $\mathbf{x} = [x_1, x_2, ..., x_n]$ refers to the design variable vector, which is the only component of $\mathbf{I}$ used for calculation of the objective function, where n represents the total number of design variables.

The other two components $\boldsymbol{\sigma}$ and $\boldsymbol{\alpha}$ in Equation (3.2) constitute the strategy parameter set of $\mathbf{I}$. As the search carries on, these parameters automatically adjust themselves to suitable values according to topological features of the design space for a successfully implemented optimization process. This capability of ESs is characterized with the term "self-adaptation" by Schwefel [32] , and it plays a major role in the success and efficiency of the technique.

In general, each design variable x_i is associated with a standard deviation σ_i, which controls the mutation of the variable, acting much like a step size parameter in a conventional optimization technique. The number of independent standard deviations n_σ employed can be between 1 and n_c, i.e. $1 \leq n_\sigma \leq n_c$. In cases when $n_\sigma = 1$, a single standard deviation is used to control the mutation of all variables, and when $n_\sigma = n$, this task is performed using a separate standard deviation for each variable. In a more general case, when $1 < n_\sigma < n$, the standard deviations $\sigma_1, ..., \sigma_{n_\sigma - 1}$ are matched with the variables $c_1, ..., c_{n_\sigma - 1}$ on a separate basis, and σ_{n_σ} is used for the remaining variables $x_{n_\sigma}, ..., x_n$ [30].

If no correlation is defined between any two continuous variables, their mutations are implemented independent of each other. Rotation angles (or correlation coefficients) $\boldsymbol{\alpha} \in [-\pi, \pi]$ are then introduced to perform the mutations of the variables in a correlated manner by relating their standard deviations. This ability of the technique is significant, and enables it to seek an advantageous search direction in the design space. Considering that correlation angles can arbitrarily be defined between any two variables, the number of correlation angles n_α employed may vary between 0 and $n(n-1)/2$, such that $n_\alpha = 0$ corresponds to a case of uncorrelated mutations, and $n_\alpha = n(n-1)/2$ allows for a complete definition of correlations between all variables.

3.4.1.2 Mutation

Mutation operator in ES is based on a normal (Gaussian) distribution which is defined by two parameters: a mean (*ξ*) and standard deviation (*σ*). The mean value (*ξ*) is set to zero, the standard deviation varies adaptively by the algorithm (σ_i'), and a design variable is mutated by mutating $\bar{x}_i$ values by

$$\bar{x}_i' = x_i + N(0, \sigma_i') \tag{3.3}$$

where $N(0,\sigma_i')$ denotes a random number drawn from a Gaussian distribution with zero mean and standard deviation σ_i'. By using Gaussian distribution here, small mutations are more likely than large ones.

As mentioned above the standard deviations (step sizes) coevolve with the solutions $\mathbf{x}$ and undergo variation, i.e., $\sigma_i \rightarrow \sigma_i'$. This in fact forms the basis for self-adaptation in ESs. In order to achieve this behavior, it is essential to modify the value of σ_i first, and then mutate the x_i values with the new σ_i' value. The rationale behind this is that a new individual $[\bar{x}_i, \sigma_i']$ is effectively evaluated twice. Primarily, it is directly evaluated for its viability during survivor selection based on its design vector $\bar{\mathbf{x}}'$. Secondly, it is evaluated for its ability to create good offspring based on its strategy parameter values $\boldsymbol{\sigma}'$. Thus an individual $[\bar{\mathbf{x}}', \boldsymbol{\sigma}']$ represents both a good $\bar{\mathbf{x}}'$ that survived selection and a good $\boldsymbol{\sigma}'$ that proved successful in generating this good $\bar{\mathbf{x}}'$ from $\mathbf{x}$ [30].
Mutation has three special cases that are uncorrelated mutation with one step size, uncorrelated mutation with n step size and correlated mutations.

3.4.1.2.1 Uncorrelated Mutation with One Step Size

A single strategy parameter, σ, is used for each individual in uncorrelated mutation with one step size. This is a result of using the same distribution to mutate each x_i. The strategy parameter, σ, is mutated with e^{Γ} where Γ is a random variable and it changes with a normal distribution having 0 mean and τ standard deviation τ. Since $N(0,\tau) = \tau.N(0,1)$, the mutation mechanism is thus specified as formulated in Equations (3.4) and (3.5).

$$\sigma' = \sigma \cdot e^{\tau \cdot N(0,1)} \tag{3.4}$$

$$x_i' = x_i + \sigma' \cdot N_i(0,1) \tag{3.5}$$

A precaution is taken to prevent the use of standard deviations that are close to zero because of their negligible effects on the average, Equation (3.6).

$$\sigma' < \varepsilon_0 \Rightarrow \sigma' = \varepsilon_0 \tag{3.6}$$

It should be underlined that the proportionality constant τ is the learning rate that is to be set by the user[30].

$$\tau \propto 1/\sqrt{n} \tag{3.7}$$

where n is the problem size.

Figure 3.2 illustrates the consequences of the mutation in two dimension. The individuals are represented as $[x, y, \sigma]$ and there is only one σ meaning that the mutation step size is the same in each direction (x and y), and the points in the search space where the offspring can be placed with a given probability form a circle around the individual to be mutated.

Figure 3.2 Mutation with n=2, n_σ=1, n_α=0 [30]

3.4.1.2.2 *Uncorrelated Mutation with n Step Sizes*

Since the fitness landscape can have a different slope in each direction, introducing the multiple step size in to the algorithm brings about an enhancement to the search capacity while exploring the design space. Multiple step size σ can be easily implemented to the representation of the individuals as $[x_1,\ldots,x_n,\sigma_1,\ldots,\sigma_n]$. Then, mutation of the strategy parameter σ_i and the continuous design variable x_i are defined as:

$$\sigma_i' = \sigma_i \cdot e^{\tau' \cdot N(0,1) + \tau \cdot N_i(0,1)} \tag{3.8}$$

$$x_i' = x_i + \sigma_i \cdot N_i(0,1) \tag{3.9}$$

The constant parameters τ, τ' are known as *learning rate* for which the following values are recommended by Schwefel [32]:

$$\tau = \frac{1}{\sqrt{2\sqrt{n}}}, \qquad \tau' = \frac{1}{\sqrt{2n}} \tag{3.10}$$

where n is the total number of design variables.
As in the case of a single mutation size, there is a lower ε for each mutation size σ_i.

$$\sigma_i' < \varepsilon \;\Rightarrow\; \sigma_i' = \varepsilon \tag{3.11}$$

Figure 3.3 illustrates the consequences of the mutation in two dimensions. However, this time the individuals are represented as $\lfloor x, y, \sigma_x, \sigma_y \rfloor$ and there is different σ in each direction meaning that the mutation step sizes can differ in each direction (*x* and *y*). The points in the search space where the offspring can be placed with a given probability form an ellipse, whose axes are parallel to the coordinate axes with the length along axis *i* proportional to the value of σ_i, around the individual to be mutated.

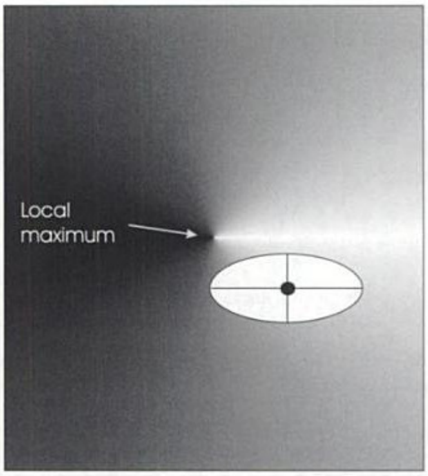


Figure 3.3 Mutation with n=2, n_σ=2, n_α=0 [30]

3.4.1.2.3 Correlated Mutations

As it is explained above, using different mutation size in each direction brings about an enhancement to the search capacity while exploring the design space. Since the axes mutation ellipsoids are parallel to the coordinate axes, design space can only be explored along the coordinate axes directions. However, search capacity of the ellipsoid can be improved by giving it the ability to rotate towards a direction of preference different from coordinate axes. This is what the correlated mutations carry out. A rotation (covariance) matrix C is introduced into the algorithm, and mutation is described by the following equations for correlated mutations.

$$\sigma_i' = \sigma_i \cdot e^{\tau' \cdot N(0,1) + \tau \cdot N\ (0,1)} \tag{3.12}$$

$$\alpha_j' = \alpha_j + \beta \cdot N(0,1) \tag{3.13}$$

$$\overline{x}' = \overline{x} + \overline{N}(\overline{0}, C') \tag{3.14}$$

where $j \in 1,..,n_\alpha$ and $n_\alpha = n(n-1)/2$. The learning rates τ and τ' are the same as uncorrelated mutation with n step sizes and $\beta \approx 5^o$ [30].

In Figure 3.4 the effects of correlated mutations in two dimensions are demonstrated. The black dot is an individual. The individuals are represented as $\lfloor x, y, \sigma_x, \sigma_y, \sigma_{x,y} \rfloor$ and the points in the search space where the offspring can be placed with a given probability form a rotated ellipse around the individual to be mutated with the axis lengths that are proportional to the σ values.

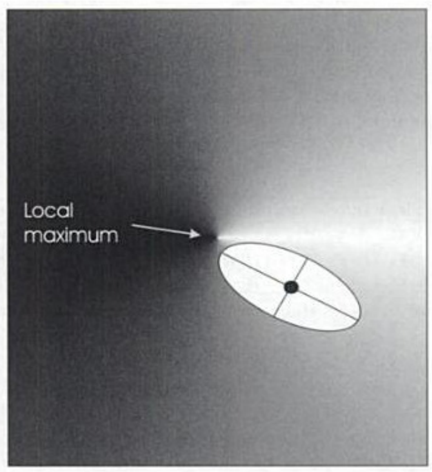


Figure 3.4 Mutation with n=2, n_σ=2, n_α=1 [30]

3.4.1.3 Recombination

Recombination is applied to create offspring population, such that μ parent individuals undergo an exchange of design characteristics to produce λ offspring individuals. A variety of distinct recombination operators exist, and in principle recombination of different components of an individual can be implemented using different operators. Assuming that s represents an arbitrary component of an individual, i.e. $\mathbf{s} \in (\mathbf{x}, \boldsymbol{\sigma}, \boldsymbol{\alpha})$, a formulation of these operators is given in Equation (3.15) as applied to produce the recombined $\boldsymbol{s}'$.

$$s_i' = \begin{cases} s_{a,i} & (1)\text{ - no recombination} \\ s_{a,i} \text{ or } s_{b,i} & (2)\text{ - discrete} \\ s_{a,i} \text{ or } s_{bj,i} & (3)\text{ - global discrete} \\ s_{a,i} + (s_{b,i} - s_{a,i})/2 & (4)\text{ - intermediate} \\ s_{a,i} + (s_{bj,i} - s_{a,i})/2 & (5)\text{ - global intermediate} \end{cases} \quad (3.15)$$

In Equation (3.15), $\boldsymbol{s}_a$ and $\boldsymbol{s}_b$ represent the $\boldsymbol{s}$ component of any two parent individuals that are chosen from the parent population at random. Accordingly, in type (1) no recombination takes place; rather $\boldsymbol{s}'$ is simply formed by duplicating $\boldsymbol{s}_a$. Type (2) refers to discrete recombination, in which each element of $\boldsymbol{s}'$ is selected from one of the two parents ($\boldsymbol{s}_a$ and $\boldsymbol{s}_b$) under equal probability. Type (3) denotes the global version of discrete recombination, such that the first parent is selected and held unchanged, while a second parent is randomly determined anew for each element of $\boldsymbol{s}$, and then s_i' is chosen from one of these two parents ($\boldsymbol{s}_a, \boldsymbol{s}_{bj}$) under equal probability. Intermediate forms of types (2) and (3) are given in types (4) and (5), respectively, which are identical to the formers except that arithmetic means of the elements are calculated.

3.4.1.4 Selection

There are mainly two different selection types known as $(\mu+\lambda)$ and (μ,λ) selections. In both schemes, individuals are selected deterministically based on their fitness scores. μ parents are selected out of λ offspring in comma type, whereas the parent population is also included to the selection operator in the plus one.

Unlike (μ,λ) variant, the $(\mu+\lambda)-\mathrm{ES}$ always comes up with a promise of guaranteed evolution. Since the parents are also involved in this variant, the parent population at any generation consists of the best μ individuals sampled thus far throughout the process. Hence, at a first sight it may seem to be more advantageous as compared to the $(\mu,\lambda)-\mathrm{ES}$. According to Bäck and Schwefel [31], [33] however, this advantage may turn into a more serious disadvantage when interpreted in view of adaptation of the strategy parameters. They argue that retreat from mis-adapted strategy parameters and local optima is more difficult in the $(\mu+\lambda)$ variant. The ratio of parent to offspring individuals (λ/μ) is generally set to a value around 5 to 7 for a satisfactory performance of the technique.

3.5 EVOLUTION STRATEGIES WITH DISCRETE DESIGN VARIABLES

At the beginning, evolution strategies were developed for continuous design spaces. However, the design of most civil engineering systems requires that the design parameters are selected from a set of predetermined values, referred to a discrete design set. For example, members in steel structures are selected from profile lists given in standards. The three different approaches (reformulations) of evolution strategies (ESs) have been proposed in the literature as extensions of the technique for solving discrete problems: Cai and Thierauf [26], Bäck and Schütz [27], and Rudolph [28]. Amongst them, the one proposed by Cai and Thierauf [26] refers to a non-adaptive reformulation of the technique and has probably found the most applications in discrete structural optimization, which were reported in Cai and Thierauf [34], Papadrakakis and Lagaros [35], Lagaros *et al.* [36], and Rajasekaran *et al.* [10]. The approach proposed by Bäck and Schütz [27] corresponds to an adaptive reformulation of the technique, which incorporates a self-adaptive strategy parameter called mutation probability. A literature survey turns up a few recent publications reporting a successful use of this approach in discrete optimum design of structural systems (Papadrakakis *et al*. [37], Ebenau *et a*l. [7]). Another adaptive reformulation of ESs is presented by Rudolph [28] for general non-linear mathematical optimization problems.

The evolution strategies for discrete design variables are same as ES for the continuous design variables except the mutation algorithm.

3.5.1 Discrete Mutation

In a discrete reformulation of ESs, an individual ($\mathbf{I}$) consists of two sets of components, which are defined as follows:

$$\mathbf{I} = \mathbf{I}(\mathbf{x},\mathbf{s}) \qquad (3.16)$$

In Equation (3.16), $\mathbf{x} = [x_1..x_i..x_n]$ stands for the design vector, and $\mathbf{s}$ represents the set of strategy parameters employed by the individual for establishing an automated problem-specific search mechanism in exploring the design space.

Every offspring individual is subjected to mutation, resulting in a new set of values for the design variables ($\mathbf{x}'$) and strategy parameters ($\mathbf{s}'$) of the individual, Equation (3.17). This implies that not only the design information, but also the search strategy of the individual is altered during this process.

$$\text{mut}\ (\mathbf{I}(\mathbf{x},\mathbf{s})) = \mathbf{I}'(\mathbf{x}',\mathbf{s}') \qquad (3.17)$$

As a general procedure, mutation of the strategy parameters is performed first. The mutated values of the strategy parameters are then used to mutate the design vector. Mutation of the design vector causes the individual to move to a new point within the design space, and can be formulated as follows:

$$\mathbf{x}' = \mathbf{x} + \mathbf{z} \tag{3.18}$$

where $\mathbf{z} = [z_1,..z_i,..z_n]$ refers to an n-dimensional random vector. The mutated design vector $\mathbf{x}' = [x_1'..x_i'..x_n']$ is simply obtained by adding this random vector to the unmutated design vector x.

3.5.2 The Approach Proposed by Bäck and Schütz

In the reformulation of technique proposed by Bäck and Schütz [27], an individual is defined as follows:

$$\mathbf{I} = \mathbf{I}(\mathbf{x}, \mathbf{s}(\mathbf{p})) \tag{3.19}$$

where $\mathbf{p} = [p_1,..p_i,..p_n]$ is referred to as the vector of mutation probability, and represents the set of adaptive strategy parameters. They are used to control (adjust) probabilities of the design variables to undergo mutation. In its most general formulation, each design variable (x_i) is coupled with a separate mutation probability (p_i), yielding n mutation probabilities in all. Nevertheless, it has been experimented that the general form suffers from a poor convergence behavior, and on the contrary the algorithm exhibits a satisfactory performance when a single mutation probability (p) is used for all the design variables of an individual (Bäck and Schütz [27]). Consequently, the number of mutation probabilities (strategy parameters) employed per individual is set to one, i.e. $\mathbf{I} = \mathbf{I}(\mathbf{x}, p)$.

Mutation is performed such that the strategy parameter p is mutated first using a logistic normal distribution, (Equation 3.20), which assures that the mutated value of p always remains within a range (0,1).

$$p' = \left(1 + \frac{1-p}{p}.e^{-\gamma.N(0,1)}\right)^{-1} \qquad (3.20)$$

In Equation (3.20), p' stands for the mutated value of p, and $N(0,1)$ represents a normally distributed random variable with expectation 0 and standard deviation 1. The factor γ here refers to the learning rate of p, and is set to the following recommended value: $\gamma = 1\big/\sqrt{2\sqrt{n}}$. Once p' is obtained from Equation (3.18), the design vector ($\vec{x}$) of the individual is mutated next as in Equation (3.19).

$$z_i = \begin{cases} 0 & , \text{ if } r_i > p' \in [0,1] \\ u_i \in \{-x_i + 1,..,n_s - x_i\} & , \text{ if } r_i \le p' \in [0,1] \end{cases} \qquad (3.21)$$

In this process, for each design variable x_i a random number r_i is generated anew in a real interval [0,1]. If $r_i > p'$, the variable is not mutated, that is $z_i = 0$ and $x_i' = x_i$. Otherwise ($r_i \le p'$), it is mutated according to a uniform distribution based variation, in which a uniformly distributed integer random number (u_i) sampled between $-x_i + 1$ and $n_s - x_i$ is assigned to z_i, whereas number of discrete values in a discrete design set. In this way, mutated value of the design variable ($x_i' = x_i + z_i$) is enforced to remain within 1 and n_s with all discrete values having an equal probability of being selected.

3.5.3 The Approach Proposed by Cai and Thierauf

In the discrete reformulation by Cai and Thierauf [26], an individual is described with a null set of adaptive strategy parameters $\mathbf{s}(\phi)$, as follows:

$$\mathbf{I} = \mathbf{I}(\mathbf{x}, \mathbf{s}(\phi)) \tag{3.22}$$

Mutation probability (p) is also employed here. Unlike the former approach, however, it is set to an appropriate static value between 0.1 and 0.4 throughout the optimization process (Cai and Thierauf 1996). This implies that every time a predefined percentage of design variables is probabilistically mutated for all the individuals, as in Equation (3.23).

$$z_i = \begin{cases} 0 & , \text{ if } r_i > p \in [0,1] \\ \pm(\kappa_i + 1) & , \text{ if } r_i \le p \in [0,1] \end{cases} \tag{3.23}$$

Again here, for each design variable x_i a random number r_i is generated anew in a real interval [0,1], and is compared with the constant mutation probability. In case of $r_i \le p$, the variable is mutated according to a Poisson distribution based variation. For this, a Poisson distributed integer random number (κ_i) is sampled first, and either a positive or negative value of $\kappa_i + 1$ is then assigned to z_i under equal probability. In Statistics, the Poisson distribution is described by the following probability function:

$$P(\kappa) = \frac{(c)^{\kappa}}{\kappa!} e^{-c}, \quad \kappa \in \{0,1,2,\ldots,+\infty\} \tag{3.24}$$

where the parameter c corresponds to both the mean and variance of the distribution. For some selected values of this parameter ($c = 3, 5, 8, 10$ and 15), a graphical representation of the probability distribution is plotted in Figure 3.5.

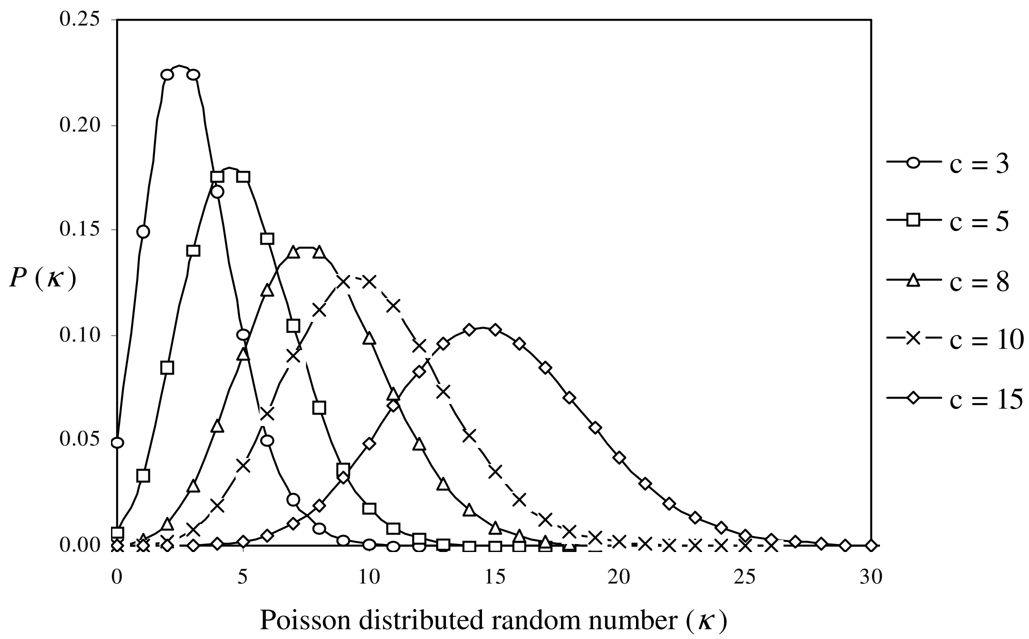


Figure 3.5 Poisson distribution for some selected values of *c*.

3.5.4 The Approach Proposed by Rudolph

Another adaptive reformulation of ESs is developed by Rudolph [28] for solving general non-linear mathematical optimization problems with unbounded integer design spaces. In this approach, mutation of a design variable is performed based on a geometric distribution in the form of

$$P(g) = \frac{1}{\psi + 1}\left(1 - \frac{1}{\psi + 1}\right)^{g}, \quad g \in \{0,1,2,..,+\infty\} \tag{3.25}$$

where g represents a geometrically distributed integer random number, and ψ corresponds to the mean (expectation) of this particular distribution. For some selected values of this parameter ($\psi = 5, 10, 20$ and 40), the variation in probability distribution pattern is shown in Figure 3.6.

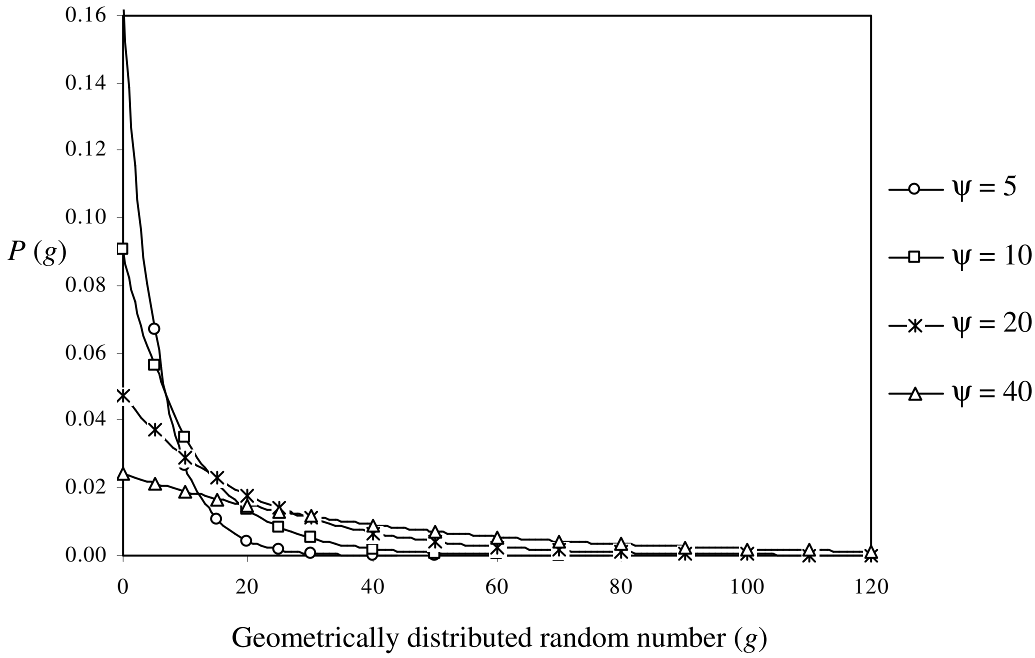


Figure 3.6 Geometric distribution for some selected values of ψ.

Rudolph's approach basically rests on a variable-wise and adaptive implementation of the parameter ψ throughout the search. The idea here is to let each variable develop a useful probability distribution pattern of its own (by adjusting ψ) for successful applications of mutation. Consequently, each design variable x_i of an individual is coupled with a different $\psi_i, i \in \{1, 2, .., n\}$ parameter, and the individual is described as follows:

$$\mathbf{I} = \mathbf{I}(\mathbf{x}, \mathbf{s}(\boldsymbol{\psi})) \tag{3.26}$$

For each design variable its strategy parameter is mutated by means of Equation (3.27).

$$\psi_i' = \psi_i . e^{\tau . N_i(0,1)} \tag{3.27}$$

In Equation (3.27), ψ_i' stands for the mutated value of ψ_i. The factor τ here refers to the learning rate of this parameter, and is set to a recommended value of $1/\sqrt{n}$ for all individuals (Rudolph [28]). Then, two geometrically distributed integer random numbers $(g_{i,1}, g_{i,2})$ are sampled using the value of ψ_i', and x_i is mutated by the difference of these two numbers, Equation (3.28).

$$z_i = g_{i,1} - g_{i,2} \tag{3.28}$$

As a final point, it is worthwhile to mention that most programming language libraries fall short of providing a function to sample the geometrically distributed numbers $g_{i,1}, g_{i,2}$. However, one can easily generate them using Equation (3.29).

$$g_{i,1}, g_{i,2} = \left\lfloor \frac{\log(1 - r_i)}{\log(1 - 1/(1 + \psi_i'))} \right\rfloor \tag{3.29}$$

3.5.5 A Reformulation of Rudolph's Approach

According to Rudolph's approach, all design variables of an individual are subjected to mutation. When interpreted in view of discrete function optimization in mathematics, this strategy is plausible, as it causes an n-dimensional mutation of the individual to a next grid point in the vicinity of the former. However, structural optimization problems are such that the overall behavior of a structural system might be very sensitive to changes in a few design variables owing to significant variations in the properties of ready sections. For a successful operation of mutation for these problems, it is essential to limit the number of design variables mutated at a time in an individual, as practiced by the former approaches. To this end, a refinement of Rudolph's approach is accomplished in Hasançebi [38], where the parameter p is incorporated and coupled with the original set of strategy parameters $\bar{\psi}$ for a harmonized implementation of the mutation operator. Accordingly, in the refined form of the Rudolph's approach, an individual is described as follows:

$$\mathbf{I} = \mathbf{I}(\mathbf{x},\mathbf{s}(\mathbf{p},\boldsymbol{\psi})) \tag{3.30}$$

In this framework, the parameter p is mutated first via Equation (3.18). Analogous to former approaches, a random number $r_i \in [0,1]$ is then generated anew for each design variable x_i and its associated strategy parameter ψ_i. If $r_i > p'$, neither x_i nor ψ_i is mutated, i.e. $\psi_i' = \psi_i$ and $z_i = 0$. If not, ψ_i is mutated first according to Equation (3.31), and is enforced to remain greater than 1.0 to preserve effectiveness of the mutation operator.

$$\psi_i' = \begin{cases} \psi_i & , \text{ if } r_i > p' \in [0,1] \\ \psi_i . e^{\tau . N_i(0,1)} \geq 1.0 & , \text{ if } r_i \leq p' \in [0,1] \end{cases} \tag{3.31}$$

Two geometrically distributed integer random numbers $(g_{i,1}, g_{i,2})$ are sampled using the value of ψ_i', and x_i is mutated by the difference of these two numbers, Equation (3.32).

$$z_i = \begin{cases} 0 & , \text{ if } r_i > p' \in [0,1] \\ g_{i,1} - g_{i,2} & , \text{ if } r_i \le p' \in [0,1] \end{cases} \tag{3.32}$$

CHAPTER 4

MINIMUM WEIGHT DESIGN PROBLEM FORMULATION OF STEEL FRAMES

4.1 DESIGN VARIABLES AND OBJECTIVE FUNCTION

For a steel structure consisting of N_m members that are collected in N_d sizing groups and N_o orientation groups for column members, the minimum weight design problem according to ASD-AISC [39] can be posed as follows:

Find a design vector **X**

$$X^T = \left[I_1, I_2, ..., I_{N_d}, O_1, O_2, ..O_{N_O}\right] \tag{4.1}$$

to minimize the weight (W) of the frame

$$W = \sum_{i=1}^{N_d} \rho_i A_i \sum_{j=1}^{N_t} L_j \tag{4.2}$$

where ρ_i and A_i are the unit weight and area of the steel section adopted for size group *i*, respectively, N_t is the total number of members in size group *i*, and L_j is the length of the member *j* which belongs to group *i*.

The design vector **X** consists of N_d integer values representing the sequence numbers of steel sections assigned to N_d member groups, as well as N_o

orientation variables corresponding the orientation of the N_o column groups. The orientation design variables are represented by the state variables 0 and 1 that uses the local axes definitions defined for elements. For a column member located along z-axis, the state variable 0 indicates that the local axis 3 of the member is directed along y-axis, whereas the state variable 1 indicates that the member's local axis 3 is directed along x-axis.

Each frame element has its own **element local coordinate system** used to define section properties, loads and output. The axes of this local system are denoted 1, 2 and 3. The first axis is directed along the length of the element; the remaining two axes lie in the plane perpendicular to the element Figure 4.1.

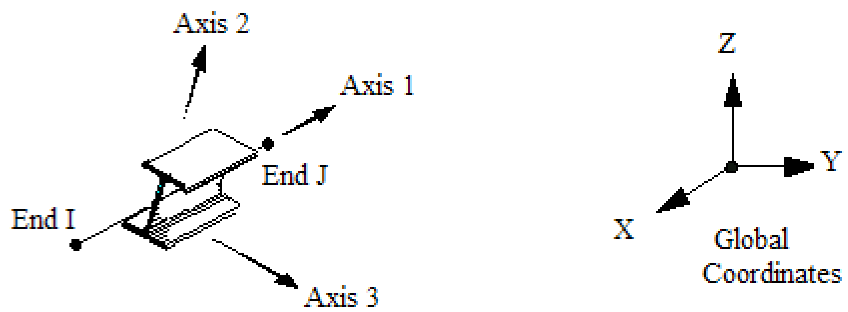


Figure 4.1 Element local coordinate system

4.2 AXIAL AND BENDING STRESS CONSTRAINTS

The members subjected to a combination of axial and flexural stresses must be sized to meet the following stress constraints:

If the member is under compression and f_a/F_a>0.15, the combined stress ratio is given by the larger of,

$$\left[\frac{f_a}{F_a}+\frac{C_{mx}\cdot f_{bx}}{\left(1-\frac{f_a}{F'_{ex}}\right)F_{bx}}+\frac{C_{my}\cdot f_{by}}{\left(1-\frac{f_a}{F'_{ey}}\right)F_{by}}\right]-1.0\leq 0 \tag{4.3}$$

$$\left[\frac{f_a}{0.60F_y}+\frac{f_{bx}}{F_{bx}}+\frac{f_{by}}{F_{by}}\right]-1.0\leq 0 \tag{4.4}$$

If the member is under compression and $f_a/F_a \leq 0.15$, a relatively simplified formula is used for the combined stress ratio,

$$\left[\frac{f_a}{F_a}+\frac{f_{bx}}{F_{bx}}+\frac{f_{by}}{F_{by}}\right]-1.0\leq 0 \tag{4.5}$$

If the member is under tension, the combined stress ratio is given by,

$$\left[\frac{f_a}{F_a}+\frac{f_{bx}}{F_{bx}}+\frac{f_{by}}{F_{by}}\right]-1.0\leq 0 \tag{4.6}$$

where, F_y is the yield stress of material. The allowable axial stress and allowable bending stress are denoted by F_a, F_b respectively. f_a stands for the computed axial stress either in compression or in tension whereas f_b is the computed normal stress in bending. The computed normal stresses about major and minor bending directions are denoted by f_{bx}, f_{by} respectively. The allowable major and minor bending stresses are represented by F_{bx}, F_{by} respectively. C_m is the moment coefficient which is added to the equations to counterbalance overestimation of the effect of secondary moments by the amplification factors ($1-f_a/F_c'$). For braced frame members there are two different C_m equations that show difference according to the existence of transverse loading between their ends. If there is no transverse loading between two ends than C_m is calculated with the following formula;

$$C_m = 0.6 - 0.4(M_1 / M_2) \tag{4.7}$$

where M_1/M_2 is the ratio of smaller end moment to the larger end moment.

$$C_m = 1 + \psi(f_a / F'_e) \tag{4.8}$$

is the formulation of C_m for the frame members having transverse loading between their ends. Ψ is a parameter that considers maximum deflection and maximum moment in the member [39].

F'_{ex} and F'_{ey} are the Euler stresses about principal axes of the member which can be formulated as,

$$F'_e = \frac{\pi^2 \cdot E}{2.5 \cdot \left[K \cdot \left(\frac{s_b}{i_b} \right) \right]^2} \tag{4.9}$$

where i_b is the radius of gyration according to the plane which is perpendicular to the bending, E is the elasticity modulus of steel, s_b is the unbraced member lengths. Finally K is the effective length K-factors about the major and minor bending directions. For beam and bracing members, K is taken equal to unity. For column members, alignment charts are furnished in ASD-AISC [39] for calculation of K values for both braced and unbraced members. In this study, however the approximate effective length formulas are given in Equation (4.10) and (4.11) are used, which are developed by Dumonteil [40]. In Hellesland [41], a verification of these formulas is conducted, where it is shown that these approximate formulas produce results which are accurate to within about -1.0 and +2.0 % of exact results.

For unbraced members:

$$K = \sqrt{\frac{1.6G_A G_B + 4(G_A + G_B) + 7.5}{G_A + G_B + 7.5}} \tag{4.10}$$

For braced members:

$$K = \frac{3G_A G_B + 1.4(G_A + G_B) + 0.64}{3G_A G_B + 2.0(G_A + G_B) + 1.28} \tag{4.11}$$

where G_A and G_B are the stiffness ratios or relative stiffness of a column at its two ends.

4.3 SHEAR STRESS CONSTRAINTS

It is also required that computed shear stresses in members are smaller than allowable shear stresses.

$$\frac{V}{A_w} \leq 0.4 \cdot F_y \cdot C_v \tag{4.12}$$

The above equation can also be expressed as follows:

$$f_v \leq F_v = 0.4 \cdot F_y \cdot C_v \tag{4.13}$$

where the computed shear stress is denoted by f_v and the allowable shear stress is abbreviated as F_v. The web shear coefficient (C_v) is taken 1.0 for rolled W-shaped members with $h/t_w \leq 2.24E/F_y$, it should be mentioned that h is the clear distance between flanges, E is the elasticity modulus, and t_w is the thickness of web. For all other symmetric shapes, C_v is calculated from Formulas G2-3, G2-4 and G2-5 in ANSI/AISC 360-05[39].

4.4 SLENDERNESS CONSTRAINTS

For the elements subjected to tension and compression, the following slenderness limitations must be satisfied according to the provisions of ASD-AISC[39],

$$\lambda_{comp} = \frac{K \cdot L}{r} \leq 200 \qquad and \qquad \lambda_{ten} = \frac{K \cdot L}{r} \leq 300 \tag{4.14}$$

As it is clear from the above formulations, the maximum slenderness ratio is set to 200 for members under compression, and it is taken as 300 for tension members. In Equation (4.14) L is the element buckling length and K is the effective length factor. Minimum radius of gyration (r) is another property of the I section member.

4.5 DISPLACEMENT AND DRIFT CONSTRAINTS

The displacement constraints are imposed such that the maximum lateral displacements are restricted to be less than $H/400$, and upper limit of story drift is set to be $h/400$, where H is the total height of the frame building and h is the height of a story.

$$\Delta \leq \frac{H}{400} \qquad and \qquad \Delta \leq \frac{h}{400} \tag{4.15}$$

4.6 GEOMETRIC CONSTRAINTS

Finally, geometric constraints between beams and columns framing into each other at a common joint can be considered for practicality of an optimum solution generated. For the two beams B1 and B2 and the column shown in Figure 4.2, one can write the following geometric constraints:

$$\frac{b_{fb}}{b_{fc}} - 1.0 \leq 0 \tag{4.16}$$

$$\frac{b'_{fb}}{\left(d_c - 2t_f\right)} - 1.0 \leq 0 \tag{4.17}$$

where b_{fb}, b'_{fb} and b_{fc} are the flange width of the beam B1, the beam B2 and the column, respectively, d_c is the depth of the column, and t_f is the flange width of the column. Equation (4.16) simply ensures that the flange width of the beam B1 remains smaller than that of the column. On the other hand Equation (4.17) enables that flange width of the beam B2 remains smaller than clear distance between the flanges of the column $(d_c - 2t_f)$.

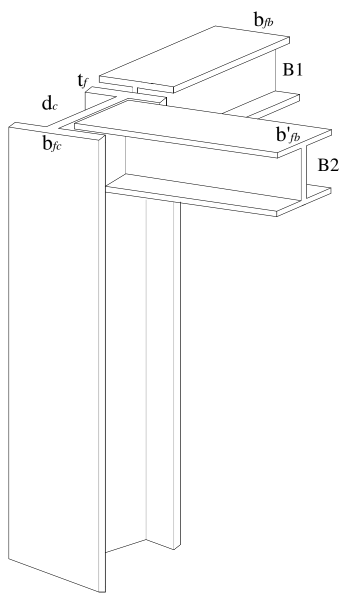


Figure 4.2 Beam-column geometric constraints

CHAPTER 5

OPTIMIZATION ALGORITHM AND SOFTWARE

In this chapter, the ES algorithm developed for optimum design of steel space frames is discussed, where size and orientation variables are used together to minimize the structural weight of the systems. The resulting algorithm is computerized in a software called OFES. The capabilities and practical features of the software are also discussed later in the chapter.

5.1 A GENERAL FLOWCHART

The general flowchart which outlines the major computational steps of the ES algorithm developed in the study is presented in Figure 5.1. Concerning this flowchart, the first two steps consist of setting the generation counter t to 0, and of creating an initial population $P(0)$. The initial population consists of μ number of parent individuals, which are customarily created through a random initialization. Hence, it is high likely that the initial population consists of a high number of unfit individuals that violate the constraints or highly overestimate the optimum. The next step is to evaluate the individuals' performances, where each individual is assigned a fitness score according to how well it satisfies the objective function and constraints of a problem at hand. In the following step, an offspring population $P'(t)$ is created through a sequential application of recombination and mutation operators to the parent population. The offspring population consists of λ number of individuals, which also undergo evaluation process (step 5) to attain fitness scores. After evaluating the offspring population, selection (step 6) is implemented to determine the survivors, which in fact form the parent population of the next generation, i.e. $P(t+1)$. A (μ,λ)-ES selection is implemented, where the parents are not included in the selection mechanism; instead selection is carried out by choosing the μ best individuals out of λ offspring in reference to the individuals' fitness scores. This completes one generation in optimization procedure, accompanied by the increase of generation counter by one (step 7). The surviving individuals in generation t make up the parent population $P(t+1)$ of the next generation. The loop between the steps 4 and 8 is iterated in the same way for each new value of the generation counter until a termination criterion is satisfied.

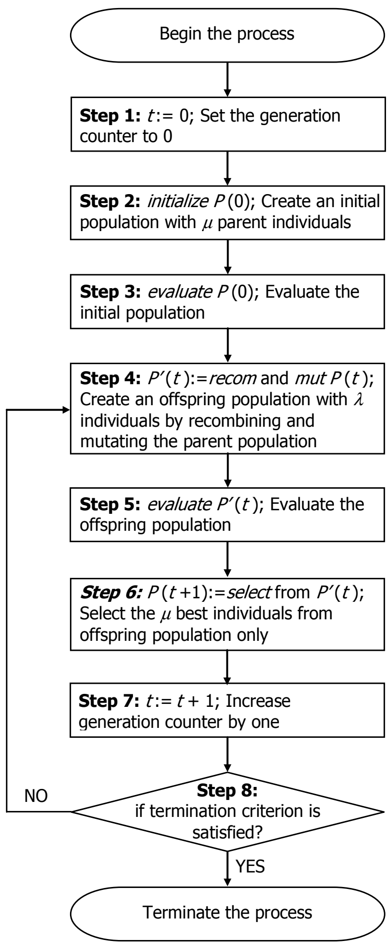


Figure 5.1 General flowchart of the ES algorithm developed in the study.

5.2 DETAILED ALGORITHM

The optimization routine discussed above forms the basic framework of the solution algorithm developed in the study. In the following subsections, computational implementations and further details of this algorithm are explained.

5.2.1 Initial Population

Initial population consists of μ number of parent solutions (individuals). Apart from a vector of size design variables $\mathbf{I}$ and a vector of orientation design variables $\mathbf{O}$, each individual incorporates three sets of strategy parameters ($p_s, p_o, \boldsymbol{\psi}$), Equation (5.1). All the strategy parameters are self-adaptive by nature, and are employed by the individual for establishing a problem-specific search scheme in an automated manner.

$$\mathbf{J} = \mathbf{J}(\mathbf{I}, \mathbf{O}, p_s, p_o, \boldsymbol{\psi}) \tag{5.1}$$

In Equation (5.1), p_s and p_o refer to the mutation probability parameters used for size and orientation variables, respectively. They are used to tune the overall mutability of the individual for size and orientation variables. The vector $\boldsymbol{\psi}$ represents the whole set of geometric distribution parameters ψ_i $(i = 1,..,N_d)$, such that each size design variable in an individual is coupled with a different ψ_i to change online the shape and flatness of the geometric distribution used to mutate it. A random initialization of population is implemented for the design vectors, and the strategy parameters are assigned to appropriate values initially ($p_s^{(0)}, p_o^{(0)} \psi_i^{(0)}$) based on numerical experimentation. In the numerical examples discussed in Chapter 6, the following initial values are used for the strategy parameter: $p_s^{(0)}$ = 0.25, $p_o^{(0)}$ =0.25 and $\psi_i^{(0)} = 20$.

5.2.2 Evaluation of Population

The initial population is measured (evaluated) next. Here, each individual is analyzed in conjunction with an external structural analysis package SAP2000 with the set of steel sections selected for the design variables and orientations assigned for column groups and force and deformation responses are obtained under the applied loads. Objective function values of the feasible solutions that satisfy all the problem constraints are directly calculated from Equation (4.2). However, infeasible solutions that violate some of the problem constraints are penalized using external penalty function approach, and their objective function values are calculated according to Equation (5.2).

$$\phi = W[1 + Penalty(J)] = W\left[1 + \alpha\left(\sum_{j=1}^{n_j}(g_j)\right)\right] \tag{5.2}$$

In Equation (5.2), ϕ is the constrained objective function value, $g_j, j = 1,..n_d$ represents the whole set of normalized constraints, and α is the penalty coefficient used to tune the intensity of penalization as a whole. Although α can be assigned to an appropriate static value, such as $\alpha = 1$, an adaptive penalty function implementation is favored by letting this parameter adjust its value automatically during the search for the most efficient optimization process (Hasançebi [42][34]). In this implementation, α is permitted to adjust itself automatically during the search, as formulated in Equation (5.3):

$$\alpha(t) = \begin{cases} (1/f).\alpha(t-1) & \text{if } \ b(t-1) \text{ is feasible} \\ f.\alpha(t-1) & \text{if } \ b(t-1) \text{ is infeasible} \end{cases} \tag{5.3}$$

where $\alpha(t)$ and $\alpha(t-1)$ denote the penalty coefficients at generations t and t-1 respectively, $b(t-1)$ is the best design at generation t-1, and f is an arbitrary constant referred to as the learning rate parameter of α. Experiments with various test problems indicate that the optimal value of f equals to 1.1.

The rationale behind Equation (5.3) is to continually enforce the algorithm to adopt a search direction along the constraint boundaries. If the best individual at the preceding generation is infeasible, the penalty is intensified somewhat in order to render the feasible regions more attractive for individuals, and thereby guiding the search towards these regions. If, however, the best individual at the preceding generation is infeasible, this time the search is directed towards infeasible regions by relaxing the penalty to some extent. The overall consequence of this action is that the search is carried out very close to constraint boundaries throughout the optimization process. As to be discussed in the numerical examples, another significant feature of the adaptive penalty function is that it avoids entrapment of the search at a local optimum, which is often the case observed when a static penalty function is utilized.

5.2.3 Recombination

After evaluated, the parent population undergoes recombination and mutation operators to yield the offspring population. Recombination provides a trade of design information between the μ parents to generate λ new (offspring) individuals. Recombination is not only to design vectors ($\mathbf{I}$ and $\mathbf{O}$), but also to the strategy parameters ($p_s, p_o, \boldsymbol{\psi}$) of the individuals in a variety of different schemes. In the present study a global discrete recombination operator is utilized for size and orientation design variables, whereas strategy parameters are recombined using intermediate scheme. Given that $\mathbf{s}$ represents an arbitrary component of an individual, i.e., $\mathbf{s} \in \{\mathbf{I}, O, p_s, p_o, \boldsymbol{\psi}\}$, the recombined $\mathbf{s}'$ can be formulated as follows:

$$s_i' = \begin{cases} s_i^a \text{ or } s_i^{bj} & \text{: global discrete} \\ s_i^a + (s_i^b - s_i^a)/2 & \text{: intermediate} \end{cases} \tag{5.4}$$

In Equation (5.4), $\mathbf{s}^a$ and $\mathbf{s}^b$ refer to the $\mathbf{s}$ component of two parent individuals which are chosen randomly from the parent population, and s_i^a and s_i^b represent typical elements of $\mathbf{s}^a$ and $\mathbf{s}^b$. In global discrete recombination, s_i^a is chosen from the two parents under equal probability such that the first parent is held unchanged, whereas the second parent is chosen a new for each element of i. In intermediate recombination scheme, both parents are kept fixed for all elements of i and their arithmetic means are calculated.

5.2.4 Mutation

Every offspring individual of the form $\mathbf{J} = \mathbf{J}(\mathbf{I}, \mathbf{O}, p_s, p_o, \boldsymbol{\psi})$ is subject to mutation, which results in a new (and expectantly improved) set of design variables ($\mathbf{I}', \mathbf{O}'$) and strategy parameters ($p_s', p_o', \boldsymbol{\psi}'$) for the individual. The application of mutation to size and orientation design variables is performed in a different manner, and is explained in the following.

5.2.4.1 Mutation of Size Variables and Strategy Parameters

The mutation of size design variables is performed as follows. First, the size mutation probability parameter p_s is mutated for an individual using a logistic normal distribution (Equation 5.5), which ensures that the mutated value of p_s always remains within a range (0,1).

$$p_s' = \left(1 + \frac{1-p_s}{p_s}.e^{-\gamma_s.N(0,1)}\right)^{-1} \tag{5.5}$$

A random number r is then generated between the range [0,1] for each size design variable I_i and its associated ψ_i parameter. If $r > p_s'$, neither I_i nor ψ_i is mutated. Otherwise, ψ_i is mutated according to a lognormal distribution based variation (Equation 5.6).

$$\psi_i' = \psi_i.e^{\tau.N_i(0,1)} \geq 1.0 \ \text{ if } r \leq p_s' \in [0,1] \tag{5.6}$$

Finally, two geometrically distributed integer random numbers $(z_{i,1}, z_{i,2})$ are sampled using the distribution parameter ψ_i', and I_i is mutated by the difference of them, Equation (5.7).

$$I_i' = I_i + z_{i,1}(\psi_i') - z_{i,2}(\psi_i') \tag{5.7}$$

In Equations (5.5-5.7), p_s', ψ_i' and I_i' are the mutated values of their corresponding parameters, $N(0,1)$ is a normally distributed random variable with expectation 0 and standard deviation 1, γ_s and τ are the learning rate constants for p_s and ψ_i, which are usually set to recommended values of $\gamma_s = 1\Big/\sqrt{2\sqrt{N_d}}$ and $1\Big/\sqrt{N_d}$, respectively where N_d stands for the number of size design variables.

5.2.4.2 Mutation of Orientation Variables and Strategy Parameters

Again, for the mutation of orientation design variables, first strategy parameter p_o is mutated using a logistic normal distribution (Equation 5.8).

$$p'_o = \left(1 + \frac{1-p_o}{p_o}.e^{-\gamma_o.N(0,1)}\right)^{-1} \tag{5.8}$$

Similar to the definitions given for Equations (5.5-5.7), in Equation (5.8), p'_o represents the mutated value of p_o, $N(0,1)$ is a normally distributed random variable with expectation 0 and standard deviation 1, and γ_o is the learning rate constants for p_o, which is set to the value $\gamma_o = 1 \big/ \sqrt{2\sqrt{N_o}}$, where N_o stands for the number of orientation design variables employed for column groups.

For each orientation variable O_i, a random number r is generated between the range [0,1]. If $r > p'$, the variable is not mutated. Otherwise, O_i is mutated as follows:

$$o'_i = switch(o_i) \text{ if } r \leq p'_o \in [0,1] \tag{5.9}$$

The $switch(o_i)$ operator in Equation (5.9) replaces the current value of the state variable with the other character in the set, i.e., if $switch[(O_i = 1)] = 0$ or $switch[(O_i = 0)] = 1$. As a result of this, the orientations of the columns belonging to i-th column group are changed as a whole.

5.2.5 Selection

Selection is implemented next to determine the survivors out of parent and offspring populations. The (μ, λ)-selection strategy is applied here, where the parents are all left to die out, and the best μ offspring having the lowest objective function scores are selected deterministically out of λ offspring. The selected (surviving) individuals become the parents of the next generation.

5.2.6 Termination

The steps 5.2.2 through 5.2.5 are implemented over a predefined number (N_{gen}) of generations.

5.3 "OFES" SOFTWARE

The solution algorithm presented above with ESs method is computerized in a design and optimization software called "OFES (Optimization of Steel Frames with Evolution Strategies" compiled in Borland Delphi source code. The opening screen of the OFES is shown in Figure 5.2. The software is automated to interact with SAP2000 v7.4 structural analysis program for generating and screening the structural models of the problems under consideration as well as carrying out a displacement based finite element analysis for each solution sampled during optimization process.

5.3.1 The Use of Software

In the following the use of the software OFES is introduced.

1. The geometric model of the structural system to be optimized is first generated in SAP2000 v7.4 (Figure 5.3). The material characteristics, structural geometry, restraints, loads, load combinations, member releases, joint and element local axes and orientations for the members other than columns are all

defined using SAP2000. It should be noted that initial frame sections can be assigned to members, and column groups can also be defined with initial orientations. However, such data will be overwritten when the optimization process is initiated.

2. After preparation of the model, “OFES Input File” command which is in “File” menu is actuated and “OFES-3D Define Data” window is brought to the view (Figure 5.4). The user defines the optimization method’s parameters as well as the structural system parameters and properties like member groups, profile lists, member properties.

3. Afterwards “Constraint” command which is under “File” menu is actuated and “OFES-3D Define Constraints” window is opened (Figure 5.5). The code and the checks that will be conducted during the process are chosen from this window.

4. “Project” menu is used to start, to stop temporarily or to end the optimization operation. These commands can also be given with the shortcut buttons on the toolbar.

5. Each of the section and orientation variables which are affected during the optimization process are monitorized in the “Design Variables” group box that is on the main menu (Figure 5.2). Furthermore on the “Current Design” group box menu, some of the information about the current design, like the weight, the volume and the satisfaction of constraints are monitorized.

6. At any time of the optimization process, the user can stop the process and see the best result which is obtained thus far and changes of optimization parameters and save this information and report it. All of these commands can be actualized from “OFES-3D Project Output” which is under “Report” menu (

Figure 5.6). In addition, the changes in optimization parameters can be monitorized from "Watch Parameters" group box throughout the iterations without stopping the process (Figure 5.2).

7. In accordance to the request, with the help of the "View" menu SAP2000 is employed and the best design is extrude viewed (Figure 5.7).

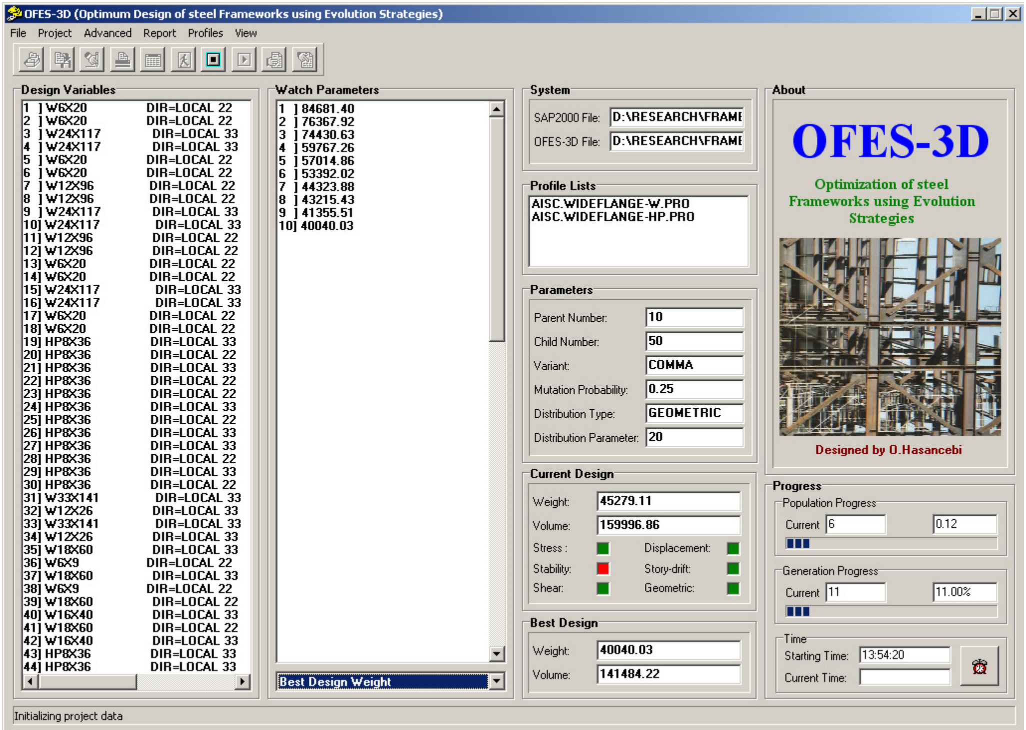


Figure 5.2 Opening screen of OFES

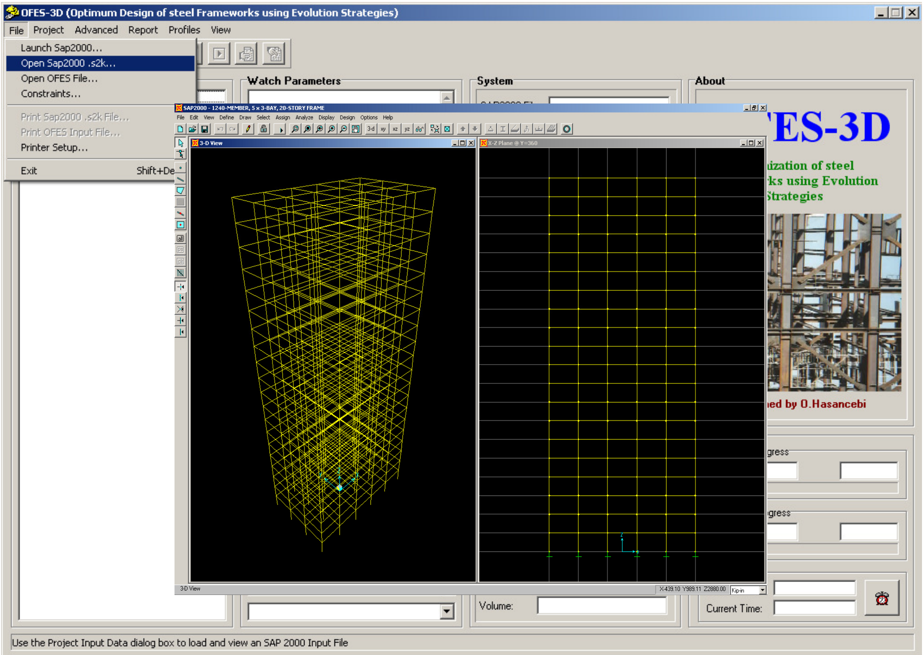


Figure 5.3 The generation of geometric model internally using SAP2000 v7.4 from OFES.

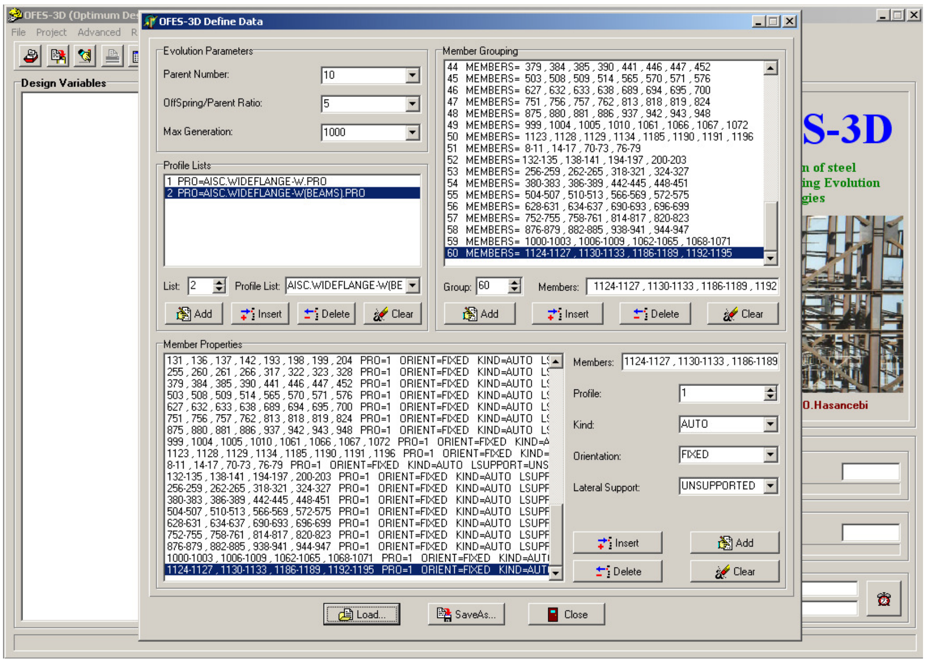


Figure 5.4. “OFES-3D Define Data” window.

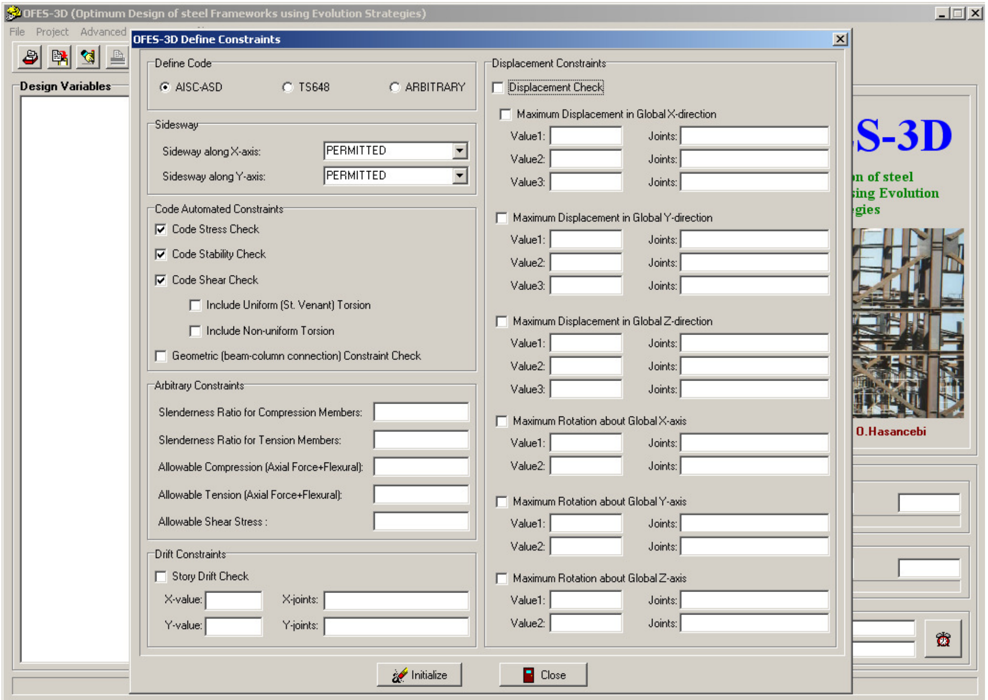


Figure 5.5“OFES-3D Define Constraint” window.

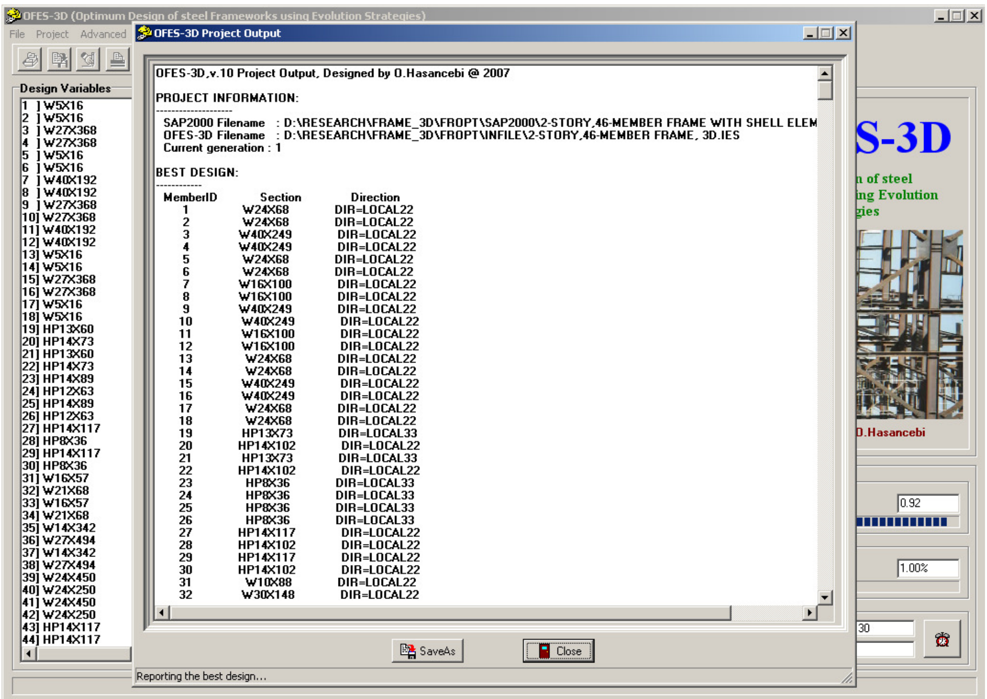


Figure 5.6“OFES-3D Project Output” window.

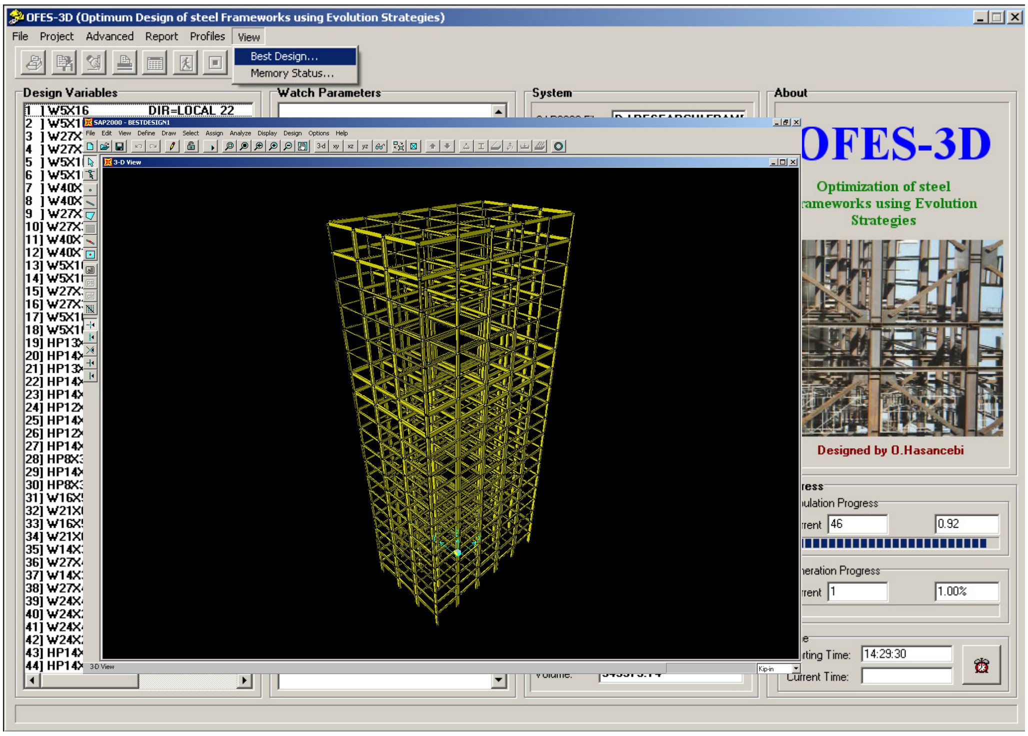


Figure 5.7 Extruded view of the best design.

5.3.2 Practical Features of Ofes

OFES has the following practical features:

1. Structural designs can be performed according to TS648 and ASD-AISC design provisions.
2. Steel structural systems can be modeled in both 2 and 3 dimension.
3. Optimum design can be performed according to the following checks which take place in the valid codes:
   - Bending and axial stress checks
   - Slenderness ratio check
   - Maximum displacement check
   - Story drift check.
   - Shear stress check

- Beam column connection compliance check.

4. The following conditions can be considered during the design of structural systems in OFES:

- Statical analysis of the structure under single and combined loads.
- Dynamic analysis of the structure with response spectrum and mode superposition methods.
- Modeling of slabs with shell elements in 2 dimensional finite element models.
- Modeling of hinged and semi-rigid column and beam connections.
- Modeling of elastic soils with springs.
- Employment of rigid diaphragms for each floor in their own planes.

5. The following practical conditions can be considered during design of structural systems in OFES:

- Optimum designs can be obtained for both of the systems that sidesway is prevented or permitted.
- All types of diagonal connection members (X, V-type, etc.) can be defined to carry the lateral loads.
- It is possible to define desired number of groups and each of the members in the same group may have the same section and orientation.
- Allowable bending stresses of the beam members can be calculated to the consideration of laterally supported to torsion or not.
- All profile lists which are prescribed in Eurocode and AISC can be used in solutions.
- Column orientations can be defined as design variables and so the required column orientations can be assigned for optimum design.

CHAPTER 6

DESIGN EXAMPLES

6.1 INTRODUCTION

The effect of column orientation on minimum weight design of steel structures is investigated using four numerical examples designed according to the provisions of ASD-AISC specification. In these examples, first steel structures are sized for minimum weight considering the size design variables only, where orientations of the column members are initially assigned and kept unchanged during optimization process. Next, the weight optimum design of structures are implemented using both size and orientation design variables. General properties of design examples are tabulated in Table 6-1.

Table 6-1 General properties of design examples

Design Examples	960 Member Steel Frame	568 Member Steel Frame	1230 Member Steel Frame	3590 Member Steel Frame
Number of stories	10	10	10	30
Types of members	Column and beam	Column and beam	Column, beam and bracing	Column, beam and bracing
Plan view of the system	Square	Square	Rectangular	Non-symmetrical
Design loads	Gravity and wind loads	Gravity and wind loads	Gravity and wind loads	Gravity and wind loads
Number of size variables	45	33	50	129
Number of orientation variables	7	7	7	16
Number of analyses for size variables	3	3	3	3
Number of analyses for size and orientation variables	5	5	5	5

6.2 DESIGN LOADS

Load cases and combinations are determined according to the ASCE 7-05, (Minimum Design Loads for Buildings and Other Structures) [43]. The structures are subjected to both gravity (GL) and lateral (LL) loads. Gravity loads include dead, live and snow loads and they are defined identically for all of the structures considered in the study. As for the lateral loading, the structure is subjected to horizontal wind forces. The gravity and lateral forces are combined under nine loading conditions which are tabulated in Table 6-2.

Table 6-2 Load Combinations

Combination	Load Cases
1	1.0GL + 1.0LL-x
2	1.0GL + 1.0LL-y
3	1.0GL + 0.75LL-x + M_T
4	1.0GL + 0.75LL-x - M_T
5	1.0GL + 0.75LL-y + M_T
6	1.0GL + 0.75LL-y - M_T
7	1.0GL + 0.75LL-x + 0.75LL-y
8	1.0GL + 0.563LL-x + 0.563LL-y + M_T
9	1.0GL + 0.563LL-x + 0.563LL-y - M_T

The load combinations are defined according to the cases that are stated in ASCE 7-05 [43] which are shown in Figure 6.1. In case 1, full design wind pressure acting on the projected area perpendicular to each principal axis of the structure is considered separately along each principal axis. For the 2nd case, 75% of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment is considered separately for each principal axis. In the 3rd case 75% of wind loading defined in case 1 is exerted to the structure simultaneously. For the last case, 75% of wind loading defined in case 2 is exerted to the structure simultaneously [43]. For 568 member steel frame nine of the combinations are taken into considerations whereas for the remaining design examples only the first two combinations are taken into consideration.

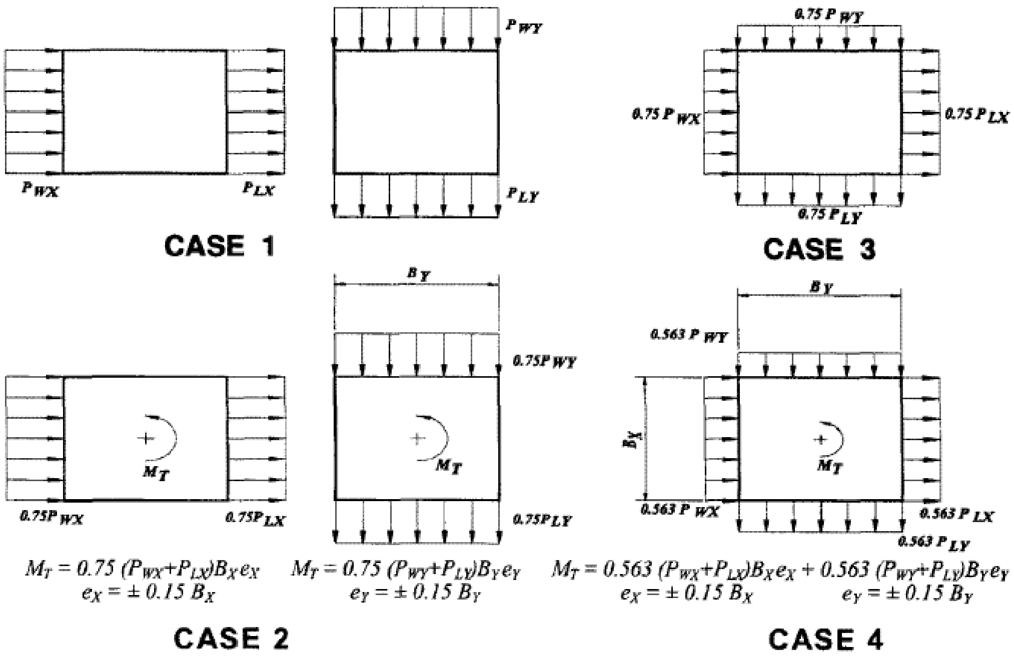


Figure 6.1 Design wind load cases [43]

6.2.1 Gravity Loads

The gravity loads acting on floor slabs cover dead (DL), live (LL) and snow (SL) loads. All the floors excluding the roof are subjected to a design dead load of 2.88 kN/m^2 (60.13 lb/ft^2) and a design live load of 2.39 kN/m^2 (50 lb/ft^2).

The roof is subjected to a design dead load of 2.88 kN/m^2 (60.13 lb/ft^2) plus snow load. The design snow load is computed using the following equation in ASCE 7-05 [43] :

$$p_s = 0.7 C_s C_e C_t I p_g \tag{6.1}$$

where p_s is the design snow load in kN/m^2, C_s is the roof slope factor, C_e is the exposure factor, C_t is the temperature factor, I is the importance factor, and p_g is the ground snow load. For a heated building having a flat and fully exposed roof, these factors may be chosen as follows: $C_s = 1.0$, $C_e = 0.9$, $C_t = 1.0$, $I = 1.0$, and $p_g = 1.20$ kN/m2 (25 lb/ft^2), resulting in a design snow load of 1.20 kN/m^2 (25 lb/ft^2). The calculated gravity loads are applied as uniformly distributed loads on the beams using distribution formulas developed for two way slabs.

6.2.2 Lateral Wind Loads

The design wind loads are also computed according to ASCE 7-05[43] using the following equations in two unit systems:

$$p_w = (0.613 K_z K_{zt} K_d V^2 I)(GC_p) \quad \text{in N/m}^2 \tag{6.2}$$

$$p_w = (0.00256 K_z K_{zt} K_d V^2 I)(GC_p) \quad \text{in lb/ft}^2 \tag{6.3}$$

where p_w is the design wind pressure, K_z is the velocity exposure coefficient, K_{zt} is the topographic factor, K_d is the wind direction factor, V is the basic wind speed, G is the gust factor, and C_p is the external pressure coefficient.

The velocity exposure coefficient (K_z) for a story is calculated using one of the two formulas given in Equations (6.4) and (6.5) based on the elevation of the story.

$$\text{For } z<15\text{ft} \qquad K_z = 2.01*(15/z_g)^{(2/alpha)} \tag{6.4}$$

$$15\text{ft} \le z \le z_g \qquad K_z = 2.01*(z/z_g)^{(2/alpha)} \tag{6.5}$$

where z_g is the nominal height of the atmospheric boundary layer used in the standard and $alpha$ is the 3-s-gust-speed power law exponent and both of the variables are taken from Table 6.2 in [43]. It should be underlined that Equations (6.4) and (6.5) are used to find the design wind pressure on windward face, along which the wind pressure increases with height. On the leeward face, the magnitude of the negative wind pressure is assumed to be constant. To calculate p_w on leeward face, K_z term in Equation (6.4) or (6.5) is replaced with K_h, which is calculated from Equation (6.6)

$$K_h = 2.01*(h/z_g)^{(2/alpha)} \tag{6.6}$$

where h is the total height of the building.

Assuming that the structures are located in an urban area with a basic wind speed of $V = 46.94$ m/s (105 mph) and exposure category B, the following values are used for these parameters: $K_{zt} = 1.0$, $K_d = 0.85$, $I = 1.0$, $G = 0.85$, $z_g = 1200 ft$, $alpha = 7$ and $C_p = 0.8$ for windward face and -0.5 for leeward face,. The calculated wind loads are applied as uniformly distributed lateral loads on the external beams of the frames located on windward and leeward facades at every floor level.

6.3 PROFILE LIST AND MATERIAL PROPERTIES

The wide-flange (W) profile list consisting of 297 ready sections is used to size column members in the design examples. On the other hand, beams and diagonals are selected from discrete sets of 171 and 147 economical sections selected from wide-flange profile list based on area and inertia properties in the former, and on area and radii of gyration properties in the latter. In all the design examples, the following material properties of the steel are used: modulus of elasticity (E) = 29000ksi (203893.6MPa) and yield stress (F_y) = 36ksi (253.1MPa).

6.4 ES PARAMETERS

In all the design examples, the parameters of the discrete evolution strategy method employed are set to the following values for an efficient optimum design process: population parameters ($\mu = 10$, $\lambda = 60$), initial values of the strategy parameters ($p_s^{(0)} = 0.25$, $p_o^{(0)} = 0.25$, $\psi_i^{(0)} = 20$), and maximum generation number ($N_{gen} = 1000$).It follows that a total of 60000 structural analyses are performed in each example to obtain the optimum designs reported in the paper.

6.5 DESIGN EXAMPLE 1: 960 MEMBER STEEL FRAME

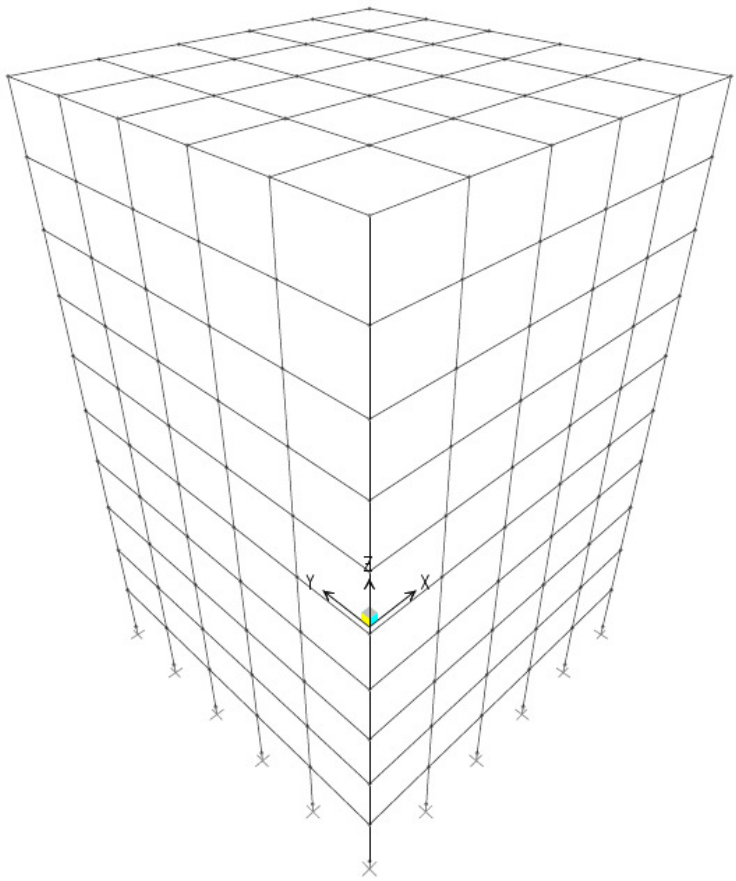
In this example it is intended to examine the effect of intensity of lateral loading on optimal layout of column orientations. A 10 story 5x5 bay regular steel space frame consisting of 960 members is taken as the base for the application (Figure 6.2). The structure is designed under four different wind force cases such that in addition to a regular basic wind speed of 105mph, the building is subjected to wind forces calculated based on basic wind speeds of 90, 120 and 150 mph. The gravity loading and initial column orientations are defined the same in all the test cases. Two design studies are carried out for each test case depending on the type of design variables included in the optimization process. First, the structure is designed for minimum weight considering the size design variables only, where initial orientations of the column members are kept unchanged. Second, size and orientation design variables are employed together to minimize the weight of the frame. The results obtained in each case are compared and optimal layout of column orientations is investigated.

6.5.1 Structural System

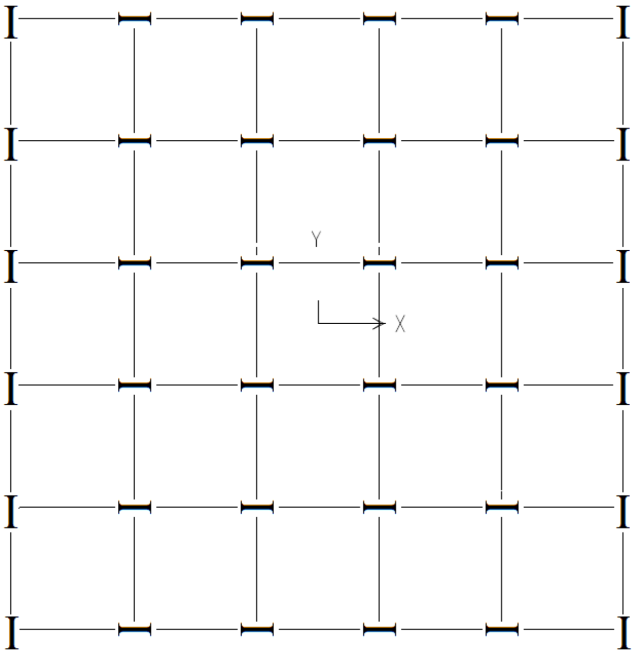
The 10-story steel space frame consists of 960 members and 396 joints. It has five bays in x-direction and five bays in y-direction with a regular bay spacing of 15ft (4.57m), and the story height is equal to 12ft (3.66m). Hence, the structural system occupies a space of $75x75ft^2$ ($522.58m^2$) in plan and 120ft (36.58m) in elevation. Figure 6.2 shows 3D, plan and side views of this structure. The stability of the structure is provided with moment resisting connections and columns are fixed to the foundation.

To satisfy practical fabrication requirements, 960 members of the frame are collected under 45 member size groups (size design variables); 35 column size groups and 10 beam size groups. The member groups are clearly tabulated in Table 6-3. The grouping of the members is performed both in plan and elevation levels. In plan level, columns are collected under seven groups considering the symmetry of the structure about x and y axes as shown in Figure 6.3. Four corner columns are placed in first group. The second and third groups are the outer columns in x-z direction and in y-z direction, respectively. Inner columns are divided into four groups that are inner corner columns (4th), inner x-z direction columns (5th), inner y-z direction columns (6th) and the central columns (7th). On the other hand, beams are collected under two groups as outer and inner beams in plan level grouping. In elevation level grouping, it is enabled that member groups have the same sections in every two stories

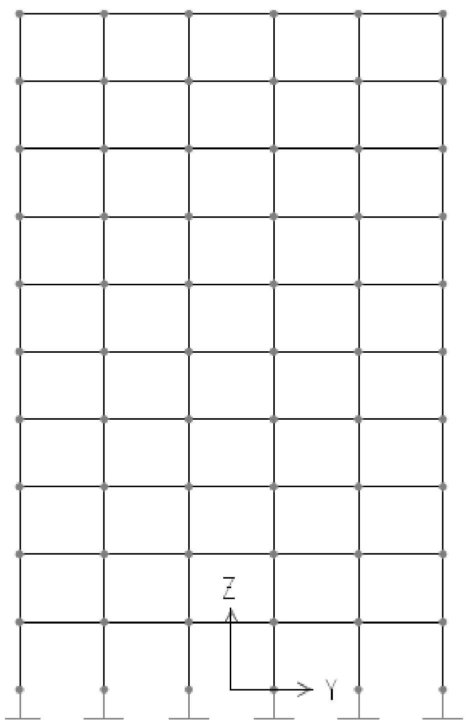
The orientation variables are assigned keeping in mind that all the successive columns along the frame height (i.e., columns that that lie on the vertical line) must have the same orientation for practical requirements. Hence, each column group shown in Figure 6.3 is associated with one orientation variable. Thus, a total of seven orientation variables are defined in the problem.



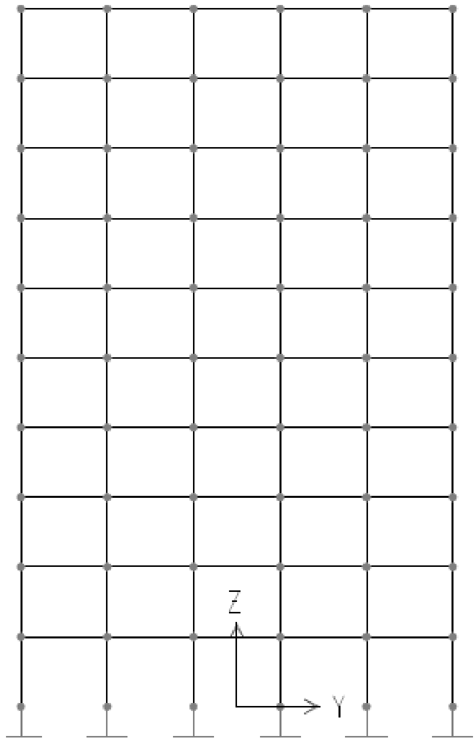
(a) 3D view



(b) Plan view



(c) Side view



(d) Side view

Figure 6.2 960 member steel frame

Load cases and combinations are determined according to the ASCE 7-05 [43], as explained in Section 6.2. In this example only one design case is studied. The 1st and the 2nd combinations that are tabulated in Table 6-2 are taken into consideration for this design case. The resulting gravity loading on the beams of roof and floors is tabulated in Table 6-4 and the wind loadings on windward and leeward faces of the frame calculated based on different basic wind speeds are presented in Table 6-5 and Table 6-6.

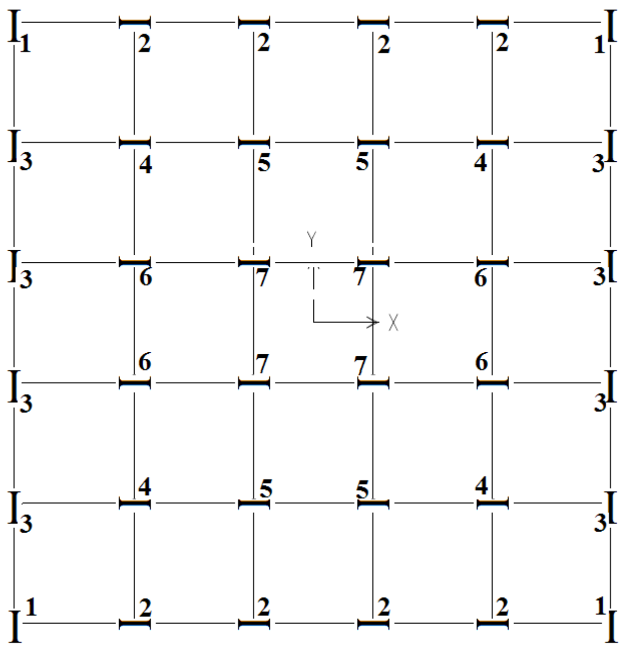


Figure 6.3 Grouping of members of 960-member steel frame in plan level.

Table 6-3 Member grouping for 960 member steel frame.

Member Group	Group Name	Member Group	Group Name
1	1st &2nd floor corner columns	24	7th &8th floor inner columns in x-z direction
2	3rd &4th floor corner columns	25	9th &10th floor inner columns in x-z direction
3	5th &6th floor corner columns	26	1st &2nd floor inner columns in y-z direction
4	7th &8th floor corner columns	27	3rd &4th floor inner columns in y-z direction
5	9th&10th floor corner columns	28	5th &6th floor inner columns in y-z direction
6	1st &2nd floor outer columns in x-z direction	29	7th &8th floor inner columns in y-z direction
7	3rd &4th floor outer columns in x-z direction	30	9th &10th floor inner columns in y-z direction

Table 6-3 (continued)

Member Group	Group Name	Member Group	Group Name
8	5th &6th floor outer columns in x-z direction	31	1st &2nd floor inner columns
9	7th &8th floor outer columns in x-z direction	32	3rd &4th floor inner columns
10	9th &10th floor outer columns in x-z direction	33	5th &6th floor inner columns
11	1st &2nd floor outer columns in y-z direction	34	7th &8th floor inner columns
12	3rd &4th floor outer columns in y-z direction	35	9th &10th floor inner columns
13	5th &6th floor outer columns in y-z direction	36	1st &2nd floor outer beams
14	7th &8th floor outer columns in y-z direction	37	3rd &4th floor outer beams
15	9th &10th floor outer columns in y-z direction	38	5th &6th floor outer beams
16	1st &2nd floor inner corner columns	39	7th &8th floor outer beams
17	3rd &4th floor inner corner columns	40	9th &10th floor outer beams
18	5th &6th floor inner corner columns	41	1st &2nd floor inner beams
19	7th &8th floor inner corner columns	42	3rd &4th floor inner beams
20	9th &10th floor inner corner columns	43	5th &6th floor inner beams
21	1st &2nd floor inner columns in x-z direction	44	7th &8th floor inner beams
22	3rd &4th floor inner columns in x-z direction	45	9th &10th floor inner beams
23	5th &6th floor inner columns in x-z direction		

Table 6-4 Gravity loading on the beams of 960 member steel frame.

Beam Type	Uniformly Distributed Load	
	Outer Beams	Inner Beams
	kN/m (lb/ft)	kN/m (lb/ft)
Roof Beams	5.54 (379.4)	11.08 (758.8)
Floor Beams	8.04 (550.65)	16.08 (1101.3)

Table 6-5 Wind Loading Values under Wind Speeds of 90mph and 105mph (in kN/m, lb/ft)

Floor	Z(ft)	V=90mph		V=105mph	
		Windward	Leeward	Windward	Leeward
1	12	1,21 (82,66)	1,37 (93,58)	1,64 (112,5)	1,86 (127,4)
2	24	1,38 (94,54)	1,37 (93,58)	1,88 (128,7)	1,86 (127,4)
3	36	1,55 (106,2)	1,37 (93,58)	2,11 (144,5)	1,86 (127,4)
4	48	1,68 (115,2)	1,37 (93,58)	2,29 (156,9)	1,86 (127,4)
5	60	1,79 (122,8)	1,37 (93,58)	2,44 (167,2)	1,86 (127,4)
6	72	1,89 (129,4)	1,37 (93,58)	2,57 (176,1)	1,86 (127,4)
7	84	1,97 (135,2)	1,37 (93,58)	2,69 (184,1)	1,86 (127,4)
8	96	2,05 (140,5)	1,37 (93,58)	2,79 (191,2)	1,86 (127,4)
9	108	2,12 (145,3)	1,37 (93,58)	2,89 (197,8)	1,86 (127,4)
10	120	1,09 (74,87)	0,68 (46,79)	1,49 (101,9)	0,93 (63,69)

Table 6-6 Wind Loading Values under Wind Speeds of 120mph and 150mph (in kN/m, lb/ft)

Floor	Z (ft)	V=120mph		V=150mph	
		Windward	Leeward	Windward	Leeward
1	12	2,15 (146,9)	2,43 (166,4)	3,35 (229,6)	3,8 (260)
2	24	2,45 (168,1)	2,43 (166,4)	3,83 (262,6)	3,8 (260)
3	36	2,76 (188,7)	2,43 (166,4)	4,3 (294,9)	3,8 (260)
4	48	2,99 (204,9)	2,43 (166,4)	4,67 (320,1)	3,8 (260)
5	60	3,19 (218,4)	2,43 (166,4)	4,98 (341,2)	3,8 (260)
6	72	3,36 (230)	2,43 (166,4)	5,25 (359,4)	3,8 (260)
7	84	3,51 (240,4)	2,43 (166,4)	5,48 (375,6)	3,8 (260)
8	96	3,65 (249,7)	2,43 (166,4)	5,7 (390,2)	3,8 (260)
9	108	3,77 (258,3)	2,43 (166,4)	5,89 (403,6)	3,8 (260)
10	120	1,94 (133,1)	1,21 (83,18)	3,04 (208)	1,9 (130)

In each of the four test cases, the wide-flange (W) profile list consisting of 297 ready sections is used to size column members and beams are selected from discrete sets of 171 economical sections selected from wide-flange profile list based on area and inertia properties. Provisions of ASD-AISC [39] are taken into consideration for the stress, stability, and geometric constraints. Displacements of all the joints in x and y directions are limited to 9.15cm (3.6in), and the upper limit of inter story drifts is set to 0.91cm (0.36in)

6.5.2 Test Results

For each case of the basic wind speeds (that is, V = 90, 105, 120 and 150 mph) a total of eight independent runs are performed considering the stochastic nature of the ES technique. First, three independent runs are conducted to minimize the weight of the frame using size variables only. Next, additional five runs are implemented where size variables are used together with orientation variables simultaneously to minimize the frame weight.

6.5.2.1 The Case for Basic Wind Speed of $V = 90$ mph

The results of the eight runs produced for the basic wind speed case of $V = 90$ mph are tabulated in Table 6-7 in terms of the minimum weight and volume of the frame attained in each run. The design history curves obtained in these runs are plotted in Figure 6.4, which show the improvement of the feasible best design during the search process in each run. The number of structural analyses performed is shown in the horizontal axis in this graph, whereas the vertical axis represents the variation of the best feasible design weights obtained thus far during the search.

It is seen from Table 6-7 or Figure 6.4, the best design weight of 465,884.61lb (211,325.26 kg) is obtained when the frame is sized for minimum weight under initial choice of column orientations. When column orientations are permitted to vary in the optimization process, the best design weight of the frame has been improved to 438,602.37lb (198,950.035kg). The sectional designations attained for member groups in both design cases are compared in Table 6-8. The optimal layout of the column orientation achieved in the latter case is shown in Figure 6.5. It follows that the optimal layout of column orientations can lead to a reduction as much as 5.9 % in the frame weight.

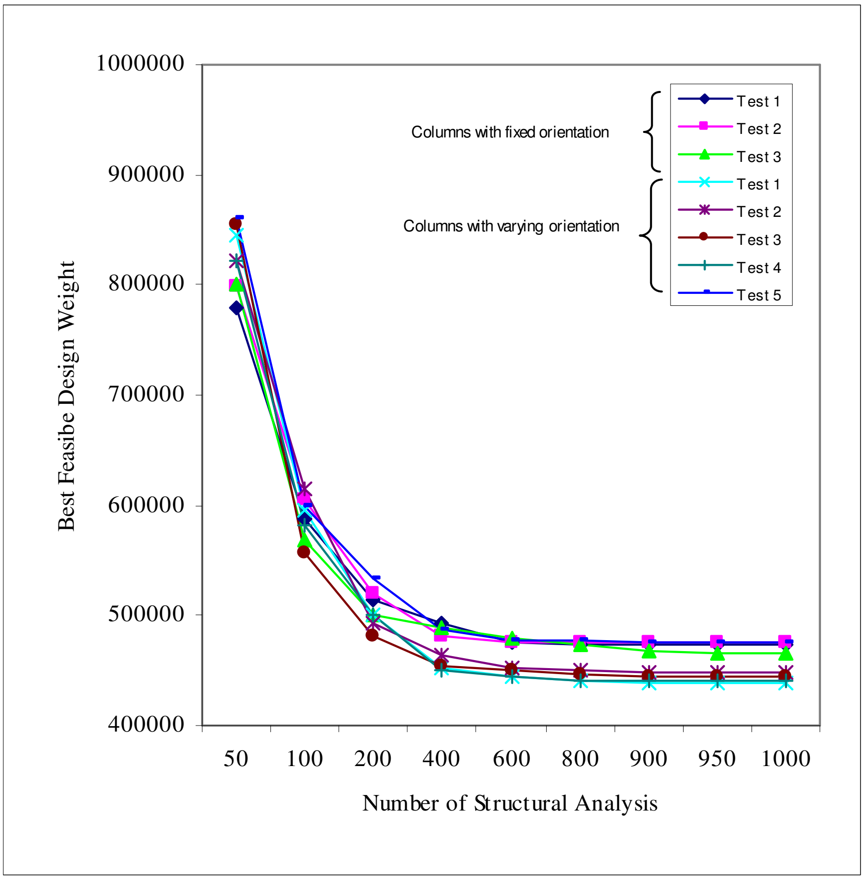


Figure 6.4 Design history graph for the basic speed wind case of $V = 90$ mph.

Table 6-7 The minimum design weights and volumes obtained for 960-member frame in eight runs for the basic wind speed case of $V = 90$ mph

	Columns with fixed orientations			Columns with varying orientations				
	Test 1	Test 2	Test 3	Test1	Test2	Test3	Test4	Test5
Total Weight	473747,71	474685	465884,6	438602,4	448927,3	444330,1	440614,3	474839,5
Total Volume	1674020,2	1677332,1	1646235,4	1549831,7	1586315,5	1570070,9	1556941	1677877,9

Table 6-8 A comparison of best designs obtained for 960-member steel frame for the basic speed wind case of $V = 90$ mph.

Group Number	Best design with initial column orientations		Best design with varying column orientations	
	Ready Section	Area (cm^2), (in^2)	Ready Section	Area (cm^2), (in^2)
1	W12X53	100,64 (15,6)	W8X48	90,97 (14,1)
2	W10X49	92,9 (14,4)	W10X45	85,81 (13,3)
3	W10X33	62,645 (9,71)	W10X33	62,645 (9,71)
4	W8X31	58,903 (9,13)	W14X34	64,52 (10)
5	W8X24	45,677 (7,08)	W8X24	45,677 (7,08)
6	W14X99	187,74 (29,1)	W12X65	123,23 (19,1)
7	W12X65	123,23 (19,1)	W10X54	101,94 (15,8)
8	W12X58	109,68 (17)	W10X49	92,9 (14,4)
9	W10X45	85,81 (13,3)	W8X35	66,45 (10,3)
10	W8X31	58,903 (9,13)	W12X26	49,355 (7,65)
11	W14X74	140,64 (21,8)	W18X76	143,87 (22,3)
12	W12X65	123,23 (19,1)	W10X60	113,55 (17,6)
13	W10X49	92,9 (14,4)	W10X49	92,9 (14,4)
14	W12X40	76,13 (11,8)	W12X40	76,13 (11,8)
15	W8X31	58,903 (9,13)	W12X26	49,355 (7,65)
16	W14X109	206,45 (32)	W14X109	206,45 (32)
17	W12X96	181,94 (28,2)	W12X96	181,94 (28,2)
18	W10X77	145,81 (22,6)	W18X76	143,87 (22,3)
19	W12X53	100,64 (15,6)	W10X49	92,9 (14,4)
20	W10X49	92,9 (14,4)	W8X28	53,226 (8,25)
21	W14X109	206,45 (32)	W21X111	210,97 (32,7)
22	W14X99	187,74 (29,1)	W14X90	170,97 (26,5)
23	W12X79	149,68 (23,2)	W12X72	136,13 (21,1)
24	W12X53	100,64 (15,6)	W10X54	101,94 (15,8)
25	W8X31	58,903 (9,13)	W8X28	53,226 (8,25)
26	W24X131	248,39 (38,5)	W14X109	206,45 (32)
27	W14X90	170,97 (26,5)	W14X99	187,74 (29,1)

Table 6-8 (cntinued)

Group Number	Best design with initial column orientations		Best design with varying column orientations	
	Ready Section	Area (cm^2), (in^2)	Ready Section	Area (cm^2), (in^2)
28	W12X72	136,13 (21,1)	W12X72	136,13 (21,1)
29	W10X49	92,9 (14,4)	W12X53	100,64 (15,6)
30	W8X28	53,226 (8,25)	W8X28	53,226 (8,25)
31	W14X120	227,74 (35,3)	W14X99	187,74 (29,1)
32	W14X90	170,97 (26,5)	W14X90	170,97 (26,5)
33	W12X72	136,13 (21,1)	W12X65	123,23 (19,1)
34	W12X65	123,23 (19,1)	W12X53	100,64 (15,6)
35	W8X31	58,903 (9,13)	W12X30	56,71 (8,79)
36	W8X18	33,935 (5,26)	W14X22	41,871 (6,49)
37	W14X22	41,871 (6,49)	W14X22	41,871 (6,49)
38	W14X22	41,871 (6,49)	W14X22	41,871 (6,49)
39	W6X16	30,581 (4,74)	W8X18	33,935 (5,26)
40	W6X15	28,581 (4,43)	W6X12	22,903 (3,55)
41	W16X26	49,548 (7,68)	W12X26	49,355 (7,65)
42	W14X26	49,613 (7,69)	W16X26	49,548 (7,68)
43	W16X26	49,548 (7,68)	W14X22	41,871 (6,49)
44	W14X22	41,871 (6,49)	W14X22	41,871 (6,49)
45	W8X18	33,935 (5,26)	W8X18	33,935 (5,26)
Weight	465,884.61 lb (211,325.26 kg)		438,602.37lb (198,950.035kg).	

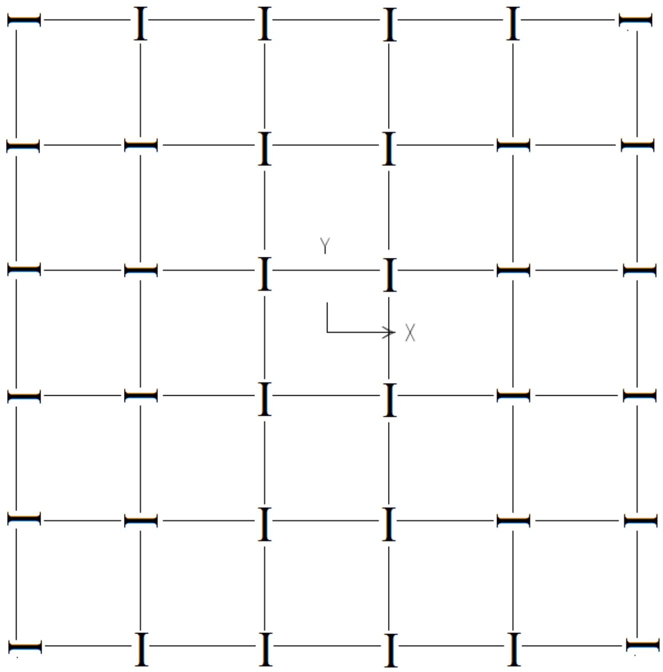


Figure 6.5 Optimal layout column orientation for 960-member steel frame for the basic speed wind case of $V = 90$ mph.

6.5.2.2 The Case for Basic Wind Speed of $V = 105$ *mph*

The results of the eight runs produced for the basic wind speed case of $V = 105$ mph are tabulated in Table 6-9 in terms of the minimum weight and volume of the frame attained in each run. The design history curves obtained in these runs are plotted in Figure 6.6. It is seen from Table 6-9 or Figure 6.6 that the best design weight of 507,432.09lb (230,171.196kg) is obtained when the frame is sized for minimum weight under initial choice of column orientations. When column orientations are permitted to vary in the optimization process, the best design weight of the frame has been improved to 473,395.61lb (214,732.25 kg). The sectional designations attained for member groups in both design cases are compared in Table 6-10. The optimal layout of the column orientation achieved in the latter case is shown in Figure 6.7. It follows that the optimal layout of column orientations can lead to a reduction as much as 6.7 % in the frame weight.

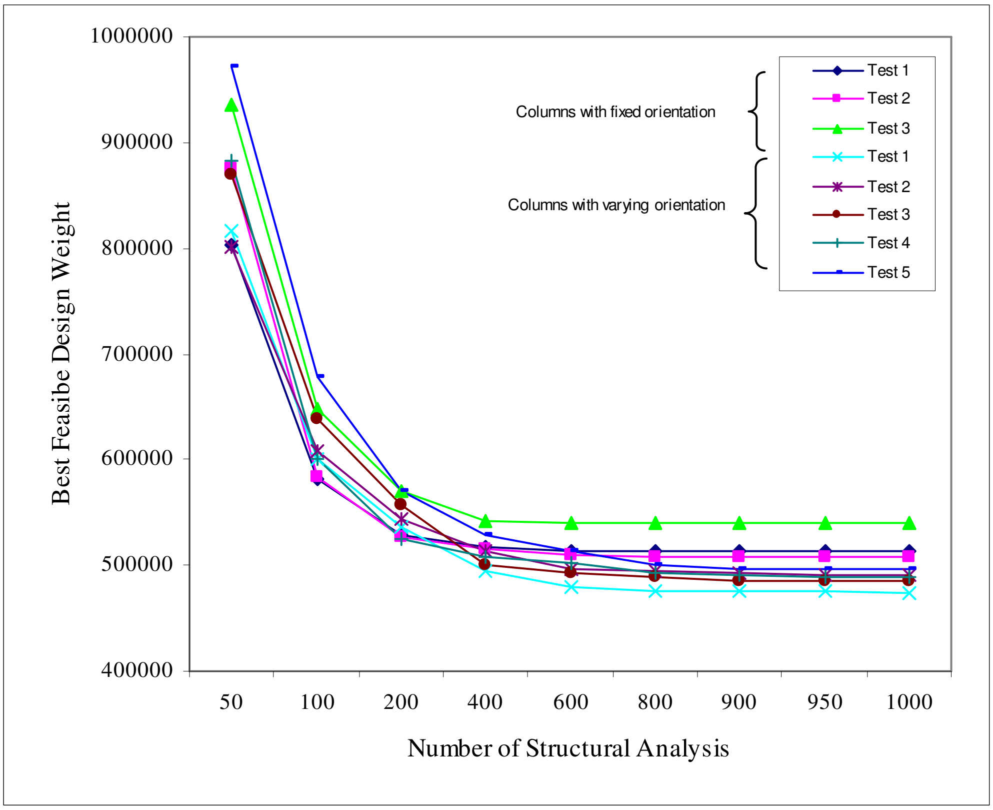


Figure 6.6 Design history graph for the basic speed wind case of $V = 105$ mph.

Table 6-9 The minimum design weights and volumes obtained for 960-member frame in eight runs for the basic wind speed case of $V = 105$ mph

	Columns with fixed orientations			Columns with varying orientations				
	Test1	Test2	Test3	Test1	Test2	Test3	Test4	Test5
Total Weight	513907,2	507432,1	539275,7	473395,6	490608,0	485069,8	489478,0	496842,7
Total Volume	1815926,4	1793046,2	1905567,8	1672776	733597,3	714027,7	1729604,2	1755627,8

Table 6-10 A comparison of best designs obtained for 960-member steel frame for the basic speed wind case of $V = 105$ mph.

Group Number	Best design with initial column orientations		Best design with varying column orientations	
	Ready Section	Area (cm^2), (in^2)	Ready Section	Area (cm^2), (in^2)
1	W8X40	75,48 (11,7)	W12X65	123,23 (19,1)
2	W10X39	74,19 (11,5)	W10X54	101,94 (15,8)
3	W10X33	62,645 (9,71)	W14X43	81,29 (12,6)
4	W12X30	56,71 (8,79)	W10X33	62,645 (9,71)
5	W12X26	49,355 (7,65)	W12X26	49,355 (7,65)
6	W14X90	170,97 (26,5)	W12X65	123,23 (19,1)
7	W12X72	136,13 (21,1)	W12X58	109,68 (17)
8	W12X65	123,23 (19,1)	W10X49	92,9 (14,4)
9	W10X49	92,9 (14,4)	W10X39	74,19 (11,5)
10	W8X31	58,903 (9,13)	W12X26	49,355 (7,65)
11	W16X67	127,1 (19,7)	W12X65	123,23 (19,1)
12	W12X65	123,23 (19,1)	W12X58	109,68 (17)
13	W10X54	101,94 (15,8)	W10X49	92,9 (14,4)
14	W10X39	74,19 (11,5)	W8X40	75,48 (11,7)
15	W8X31	58,903 (9,13)	W8X24	45,677 (7,08)
16	W14X120	227,74 (35,3)	W18X130	246,45 (38,2)
17	W10X112	212,26 (32,9)	W14X99	187,74 (29,1)
18	W12X87	165,16 (25,6)	W14X90	170,97 (26,5)
19	W12X58	109,68 (17)	W12X58	109,68 (17)
20	W8X31	58,903 (9,13)	W8X31	58,903 (9,13)
21	W24X131	248,39 (38,5)	W14X109	206,45 (32)
22	W14X109	206,45 (32)	W14X90	170,97 (26,5)
23	W12X87	165,16 (25,6)	W12X79	149,68 (23,2)
24	W12X58	109,68 (17)	W12X58	109,68 (17)
25	W8X31	58,903 (9,13)	W8X31	58,903 (9,13)
26	W36X150	285,16 (44,2)	W24X117	221,94 (34,4)

Table 6-10 (continued)

Group Number	Best design with initial column orientations		Best design with varying column orientations	
	Ready Section	Area (cm^2), (in^2)	Ready Section	Area (cm^2), (in^2)
27	W14X109	206,45 (32)	W14X99	187,74 (29,1)
28	W12X79	149,68 (23,2)	W12X79	149,68 (23,2)
29	W10X54	101,94 (15,8)	W12X53	100,64 (15,6)
30	W8X31	58,903 (9,13)	W16X40	76,13 (11,8)
31	W14X132	250,32 (38,8)	W14X109	206,45 (32)
32	W14X99	187,74 (29,1)	W14X99	187,74 (29,1)
33	W18X97	183,87 (28,5)	W12X79	149,68 (23,2)
34	W12X58	109,68 (17)	W12X53	100,64 (15,6)
35	W8X31	58,903 (9,13)	W12X30	56,71 (8,79)
36	W16X26	49,548 (7,68)	W14X22	41,871 (6,49)
37	W16X26	49,548 (7,68)	W14X22	41,871 (6,49)
38	W14X22	41,871 (6,49)	W8X18	33,935 (5,26)
39	W8X18	33,935 (5,26)	W8X18	33,935 (5,26)
40	W6X15	28,581 (4,43)	W6X16	30,581 (4,74)
41	W18X35	66,45 (10,3)	W16X31	58,839 (9,12)
42	W12X26	49,355 (7,65)	W16X31	58,839 (9,12)
43	W16X26	49,548 (7,68)	W16X26	49,548 (7,68)
44	W16X26	49,548 (7,68)	W14X22	41,871 (6,49)
45	W8X21	39,742 (6,16)	W8X21	39,742 (6,16)
Weight	507,432.09lb (230,171.20kg)		473395.61lb (214735.25kg)	

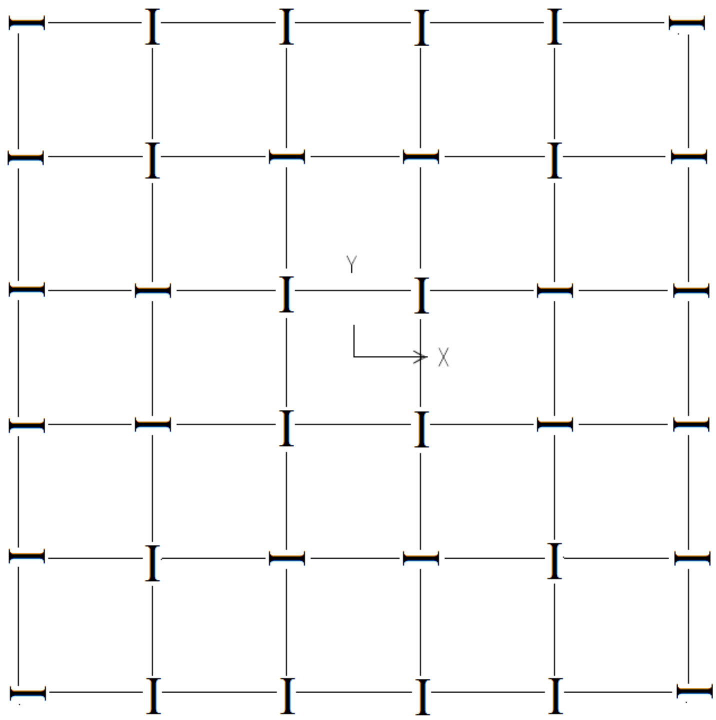


Figure 6.7 Optimal layout column orientation for 960-member steel frame for the basic speed wind cases of $V = 105$ and 120 mph.

6.5.2.3 The Case for Basic Wind Speed of $V = 120$ *mph*

The results of the eight runs produced for the basic wind speed case of $V = 120$ mph are tabulated in Table 6-11 in terms of the minimum weight and volume of the frame attained in each run. The design history curves obtained in these runs are plotted in Figure 6.8. It is seen from Table 6-11 or Figure 6.8 that the best design weight of 553,079.62 lb (250,876.92 kg) is obtained when the frame is sized for minimum weight under initial choice of column orientations. When column orientations are permitted to vary in the optimization process, the best design weight of the frame has been improved to 532,413.88lb (241,502.94kg). The sectional designations attained for member groups in both design cases are compared in Table 6-12. The optimal layout of the column orientation achieved in the latter case is exactly identical to the one obtained for the basic wind speed case of $V = 105$ (Figure 6.7). It follows that the optimal layout of column orientations can lead to a reduction as much as 3.7 % in the frame weight.

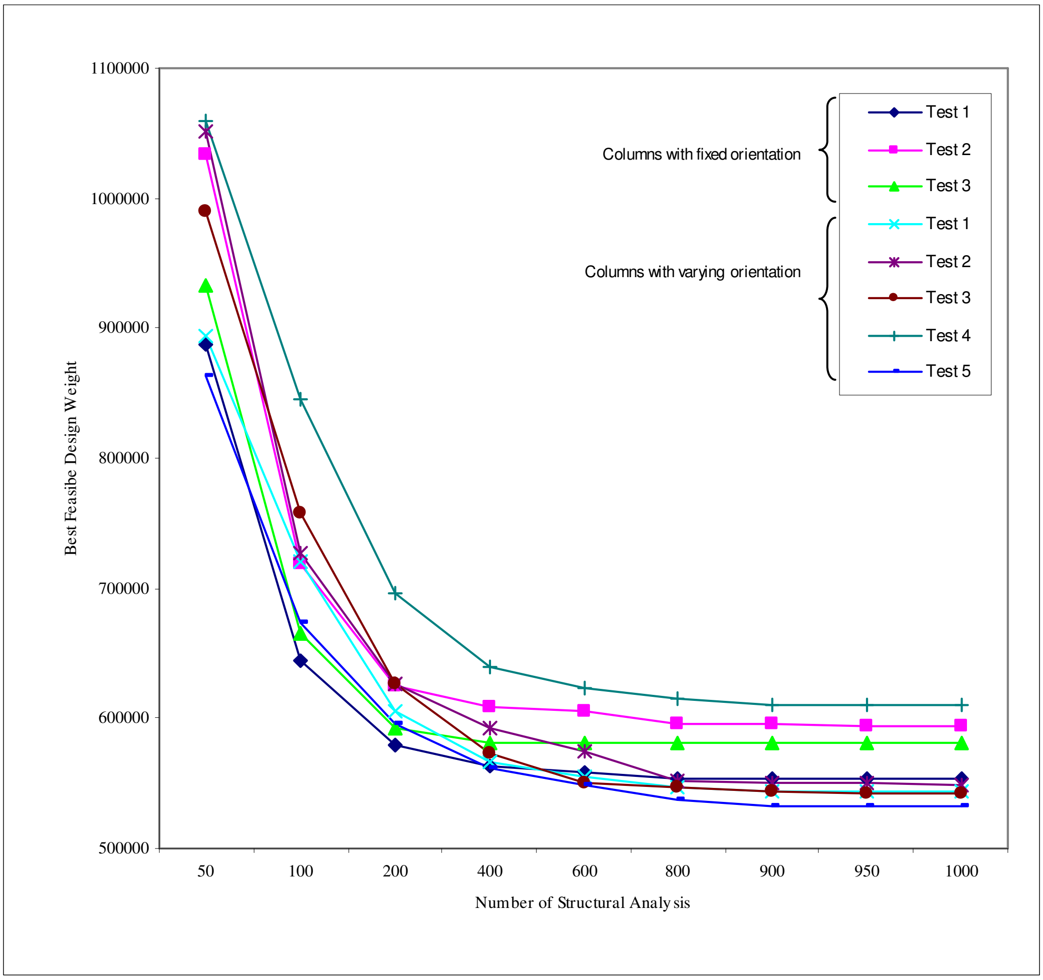


Figure 6.8 Design history graph for the basic speed wind case of $V = 120$ mph.

Table 6-11 The minimum design weights and volumes obtained for 960-member frame in eight runs for the basic wind speed case of $V = 120$ mph

	Columns with fixed orientations			Columns with varying orientations				
	Test1	Test2	Test3	Test1	Test2	Test3	Test4	Test5
Total Weight	553079,62	594861,83	580816,66	543948,32	548797	541722,04	610655,27	532413,88
Total Volume	1954345	2101985,28	2052355,7	1922078,88	1939212	1914212,2	2157792,5	1881321,1

Table 6-12 A comparison of best designs obtained for 960-member steel frame for the basic speed wind case of $V = 120$ mph.

Group Number	Best design with initial column orientations		Best design with varying column orientations	
	Ready Section	Area (cm^2),(in^2)	Ready Section	Area(cm^2), (in^2)
1	W8X48	90,97 (14,1)	W16X67	127,1 (19,7)
2	W14X48	90,97 (14,1)	W12X50	94,84 (14,7)
3	W8X31	58,903 (9,13)	W14X43	81,29 (12,6)
4	W12X26	49,355 (7,65)	W8X40	75,48 (11,7)
5	W8X21	39,742 (6,16)	W8X24	45,677 (7,08)
6	W14X90	170,97 (26,5)	W14X68	129,03 (20)
7	W12X79	149,68 (23,2)	W16X67	127,1 (19,7)
8	W12X65	123,23 (19,1)	W10X49	92,9 (14,4)
9	W10X49	92,9 (14,4)	W8X40	75,48 (11,7)
10	W8X31	58,903 (9,13)	W12X26	49,355 (7,65)
11	W18X97	183,87 (28,5)	W12X72	136,13 (21,1)
12	W12X79	149,68 (23,2)	W12X65	123,23 (19,1)
13	W16X67	127,1 (19,7)	W14X68	129,03 (20)
14	W14X48	90,97 (14,1)	W14X43	81,29 (12,6)
15	W8X31	58,903 (9,13)	W8X28	53,226 (8,25)
16	W14X132	250,32 (38,8)	W40X149	282,58 (43,8)
17	W12X106	201,29 (31,2)	W14X109	206,45 (32)

Table 6-12 (continued)

Group Number	Best design with initial column orientations		Best design with varying column orientations	
	Ready Section	Area (cm^2),(in^2)	Ready Section	Area(cm^2), (in^2)
18	W12X96	181,94 (28,2)	W12X79	149,68 (23,2)
19	W12X53	100,64 (15,6)	W27X84	160 (24,8)
20	W8X31	58,903 (9,13)	W8X31	58,903 (9,13)
21	W14X145	275,48 (42,7)	W33X152	288,39 (44,7)
22	W14X109	206,45 (32)	W14X109	206,45 (32)
23	W14X90	170,97 (26,5)	W21X111	210,97 (32,7)
24	W14X90	170,97 (26,5)	W12X65	123,23 (19,1)
25	W10X49	92,9 (14,4)	W8X35	66,45 (10,3)
26	W12X152	288,39 (44,7)	W24X117	221,94 (34,4)
27	W14X109	206,45 (32)	W14X99	187,74 (29,1)
28	W10X88	167,1 (25,9)	W12X96	181,94 (28,2)
29	W12X87	165,16 (25,6)	W12X58	109,68 (17)
30	W8X31	58,903 (9,13)	W10X33	62,645 (9,71)
31	W14X159	301,29 (46,7)	W21X122	231,61 (35,9)
32	W14X109	206,45 (32)	W14X120	227,74 (35,3)
33	W14X90	170,97 (26,5)	W14X90	170,97 (26,5)
34	W10X60	113,55 (17,6)	W12X65	123,23 (19,1)
35	W8X31	58,903 (9,13)	W8X31	58,903 (9,13)
36	W16X26	49,548 (7,68)	W14X26	49,613 (7,69)
37	W16X26	49,548 (7,68)	W14X22	41,871 (6,49)
38	W16X26	49,548 (7,68)	W14X22	41,871 (6,49)
39	W12X26	49,355 (7,65)	W14X22	41,871 (6,49)
40	W8X18	33,935 (5,26)	W6X15	28,581 (4,43)
41	W16X31	58,839 (9,12)	W16X31	58,839 (9,12)
42	W14X30	57,097 (8,85)	W21X44	83,87 (13)
43	W14X26	49,613 (7,69)	W12X26	49,355 (7,65)
44	W16X31	58,839 (9,12)	W16X26	49,548 (7,68)
45	W10X22	41,871 (6,49)	W8X21	39,742 (6,16)
Weight	553,079.62lb (250,876.92kg)		532,413.88lb (241,502.94kg)	

6.5.2.4 The Case for Basic Wind Speed of $V = 150$ ***mph***

The results of the eight runs produced for the basic wind speed case of $V = 150$ mph are tabulated in Table 6-13 in terms of the minimum weight and volume of the frame attained in each run. The design history curves obtained in these runs are plotted in Figure 6.9. It is seen from Table 6-13 or Figure 6.9 that the best design weight of 660,182.4 lb (299,458.74 kg) is obtained when the frame is sized for minimum weight under initial choice of column orientations. When column orientations are permitted to vary in the optimization process, the best design weight of the frame has been improved to 604,324.86lb (274,121.76kg). The sectional designations attained for member groups in both design cases are compared in Table 6-14. The optimal layout of the column orientation achieved in the latter case is shown Figure 6.10. It follows that the optimal layout of column orientations can lead to a reduction as much as 8.5 % in the frame weight.

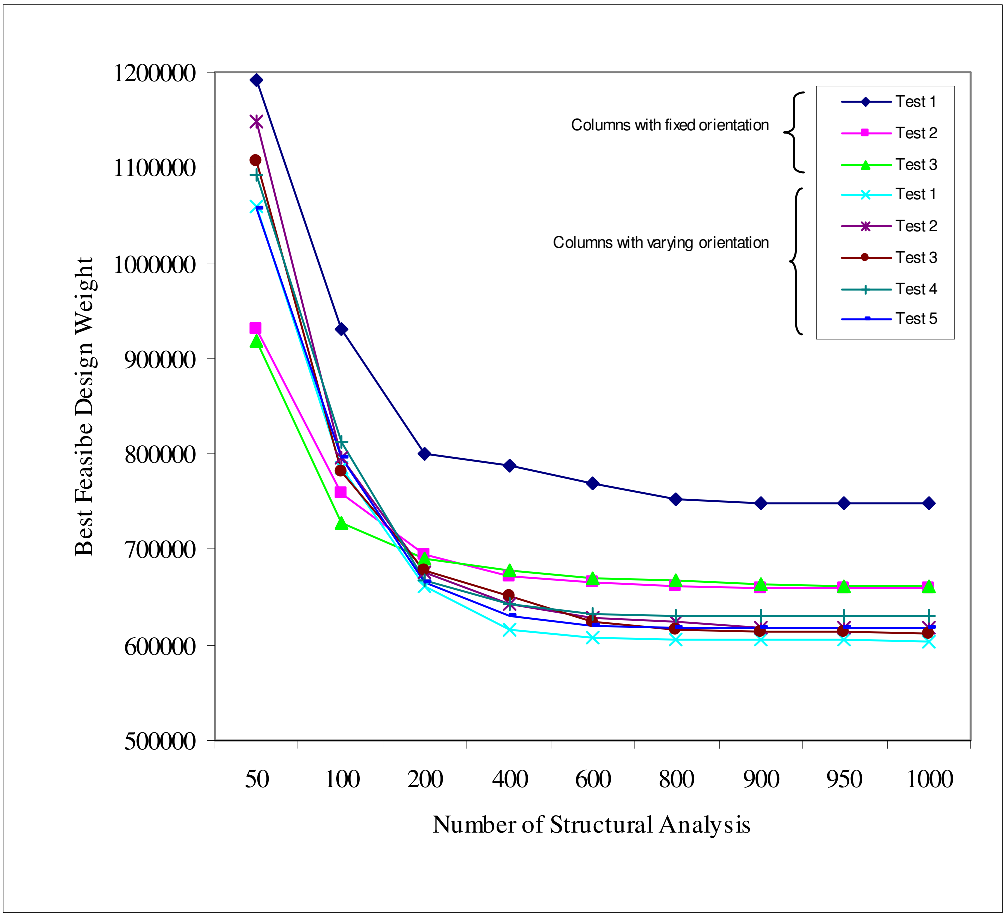


Figure 6.9 Design history graph for the basic speed wind case of $V = 150$ mph.

Table 6-13 The minimum design weights and volumes obtained for 960-member frame in eight runs for the basic wind speed case of $V = 150$ mph

	Columns with fixed orientations			Columns with varying orientations				
	Test1	Test2	Test3	Test1	Test2	Test3	Test4	Test5
Total Weight	747817,54	660182,4	662060,25	604324,86	617909,54	612631,74	631117,26	618032,2
Total Volume	2642464,8	2332800	2339435,5	2135423,52	2183425,9	2164776,5	2230096,3	2183859,4

Table 6-14 A comparison of best designs obtained for 960-member steel frame for the basic speed wind case of $V = 150$ mph.

Group Number	Best design with initial column orientations		Best design with varying column orientations	
	Ready Section	Area(cm^2) ,(in^2)	Ready Section	Area(cm^2), (in^2)
1	W21X62	118,06 (18,3)	W12X65	123,23 (19,1)
2	W10X39	74,19 (11,5)	W12X53	100,64 (15,6)
3	W16X40	76,13 (11,8)	W8X48	90,97 (14,1)
4	W14X30	57,097 (8,85)	W8X31	58,903 (9,13)
5	W8X24	45,677 (7,08)	W8X24	45,677 (7,08)
6	W14X145	275,48 (42,7)	W12X87	165,16 (25,6)
7	W24X104	197,42 (30,6)	W12X65	123,23 (19,1)
8	W12X65	123,23 (19,1)	W12X58	109,68 (17)
9	W12X65	123,23 (19,1)	W14X48	90,97 (14,1)
10	W10X45	85,81 (13,3)	W12X26	49,355 (7,65)
11	W27X94	178,71 (27,7)	W16X100	189,68 (29,4)
12	W18X86	163,23 (25,3)	W12X72	136,13 (21,1)
13	W16X67	127,1 (19,7)	W10X60	113,55 (17,6)
14	W10X49	92,9 (14,4)	W10X49	92,9 (14,4)
15	W8X31	58,903 (9,13)	W10X33	62,645 (9,71)
16	W40X192	364,52 (56,5)	W14X145	275,48 (42,7)
17	W14X132	250,32 (38,8)	W14X132	250,32 (38,8)
18	W14X109	206,45 (32)	W12X106	201,29 (31,2)
19	W14X90	170,97 (26,5)	W12X65	123,23 (19,1)
20	W12X65	123,23 (19,1)	W14X43	81,29 (12,6)
21	W12X170	322,58 (50)	W14X132	250,32 (38,8)
22	W40X192	364,52 (56,5)	W14X145	275,48 (42,7)
23	W14X99	187,74 (29,1)	W14X99	187,74 (29,1)
24	W12X79	149,68 (23,2)	W16X67	127,1 (19,7)
25	W10X45	85,81 (13,3)	W10X39	74,19 (11,5)
26	W14X176	334,19 (51,8)	W30X173	327,74 (50,8)
27	W27X146	276,77 (42,9)	W12X136	257,42 (39,9)
28	W14X145	275,48 (42,7)	W14X90	170,97 (26,5)

Table 6-14 (continued)

Group Number	Best design with initial column orientations		Best design with varying column orientations	
	Ready Section	Area(cm^2) ,(in^2)	Ready Section	Area(cm^2), (in^2)
29	W12X72	136,13 (21,1)	W18X86	163,23 (25,3)
30	W12X40	76,13 (11,8)	W8X40	75,48 (11,7)
31	W30X191	361,93 (56,1)	W27X161	305,81 (47,4)
32	W14X120	227,74 (35,3)	W12X152	288,39 (44,7)
33	W14X132	250,32 (38,8)	W14X109	206,45 (32)
34	W14X82	155,48 (24,1)	W18X76	143,87 (22,3)
35	W10X45	85,81 (13,3)	W12X40	76,13 (11,8)
36	W14X22	41,871 (6,49)	W14X22	41,871 (6,49)
37	W21X44	83,87 (13)	W10X22	41,871 (6,49)
38	W14X22	41,871 (6,49)	W8X21	39,742 (6,16)
39	W14X22	41,871 (6,49)	W14X22	41,871 (6,49)
40	W6X15	28,581 (4,43)	W5X16	30,193 (4,68)
41	W21X44	83,87 (13)	W18X46	87,1 (13,5)
42	W18X40	76,13 (11,8)	W21X44	83,87 (13)
43	W18X35	66,45 (10,3)	W18X35	66,45 (10,3)
44	W12X26	49,355 (7,65)	W14X30	57,097 (8,85)
45	W10X22	41,871 (6,49)	W10X22	41,871 (6,49)
Weight	660,182.4lb (299,458.74kg)		604,324.86lb (274,121.76kg)	

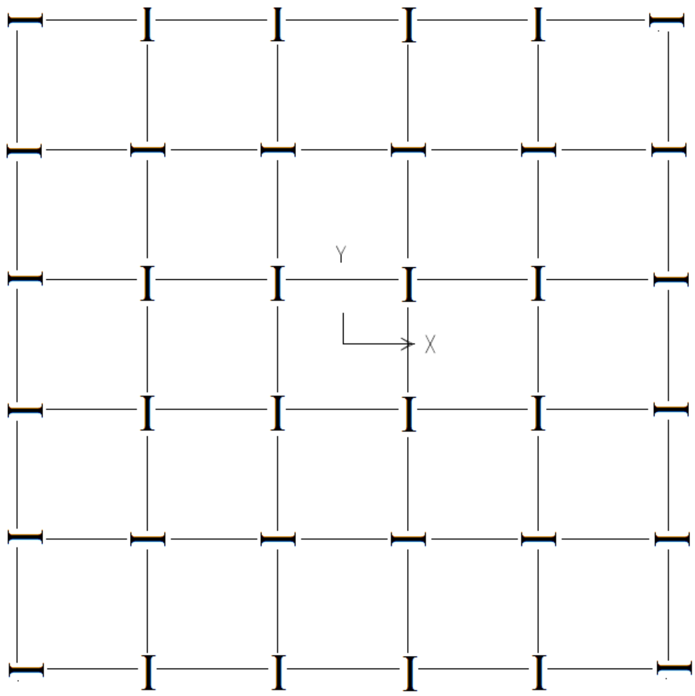


Figure 6.10 Optimal layout column orientation for 960-member steel frame for the basic speed wind cases of $V = 150$ mph.

6.6 DESIGN EXAMPLE 2: 568 MEMBER STEEL FRAME

6.6.1 Structural System

The second design example is a 10-story unbraced space steel frame consisting of 256 joints and 568 members. Figure 6.11 shows 3D, elevation and plan views of this structure. The structure consists of two parts. The first three stories (part 1) have five bays in x and y-directions, whereas the upper seven stories (part 2) have 3 bays in x and y-directions with a regular bay spacing of 20ft (6.10m). Each story has 12 ft height, and thus the total height of the structure is 120 ft (36.58m). The stability of the frame is provided with moment resisting connections and columns are fixed to the foundation.

To satisfy practical fabrication requirements, 568 members of the frame are collected under 33 member size groups; 23 column size groups and 10 beam size groups. The member groups are clearly tabulated in Table 6-15. The grouping of the members is performed both in plan and elevation levels. A plan level grouping of columns for the parts 1 and 2 are shown in Figure 6.12. On the other hand, beams are collected under two groups as outer and inner beams in plan level grouping. In elevation level grouping, it is enabled that the member groups in the three stories in the first part have the same sections, and member groups have the same sections in every two stories in the second part.

The orientation variables are assigned, bearing in mind that all the successive columns along the frame height must have the same orientation. Hence, each column group in Figure 6.12 is associated with an orientation variable, resulting in seven orientation variables in all.

The structure is analyzed for two different design cases. For the first design case the 1^{st} and the 2^{nd} combinations that are tabulated in Table 6-2 are taken into consideration. For the second design case, the structural system is designed for 9 combinations (Table 6-2). The load cases and combinations are determined according to the ASCE 7-05 [43], as explained in Section 6.2. The resulting gravity loading on the beams of roof and floors is tabulated in Table 6-16 and the wind loadings on windward and leeward faces of the frame calculated based on a basic wind speed of 105mph (46.94 m/s) is presented in Table 6-17. Torsional moment values. M_T, are calculated according to ASCE 7-05 [43] and tabulated in Table 6-18.). A joint is defined at the center of gravity of each story and the calculated torsional moment values are applied at these joints. It is crucial to mention that diaphragm constraints are also defined between all the joints of the floor systems to distribute the torsional moments equivalently.

The wide-flange (W) profile list consisting of 297 ready sections is used to size column members, and beams are selected from discrete sets of 171 economical sections selected from wide-flange profile list based on area and inertia properties. Provisions of ASD-AISC [39] are taken into consideration for the stress, stability, and geometric constraints. Displacements of all the joints in x and y directions are limited to 9.15cm (3.6in), and the upper limit of inter story drifts is set to 0.91cm (0.36in).

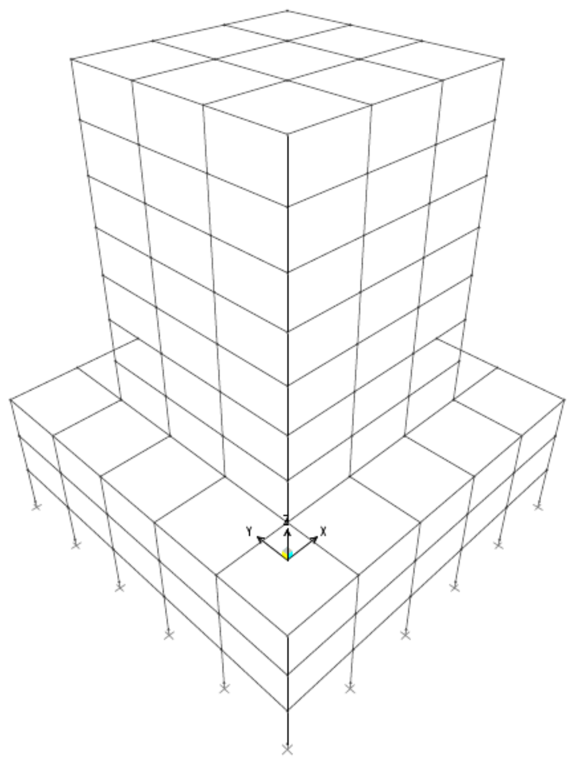
Two design cases are studied for each of the two cases. First, the structure is designed for minimum weight considering the size design variables only, where initial orientations of the column members are kept unchanged. Second, size and orientation variables are employed together to minimize the weight of the frame.

Table 6-15 Member grouping for 568 member steel frame

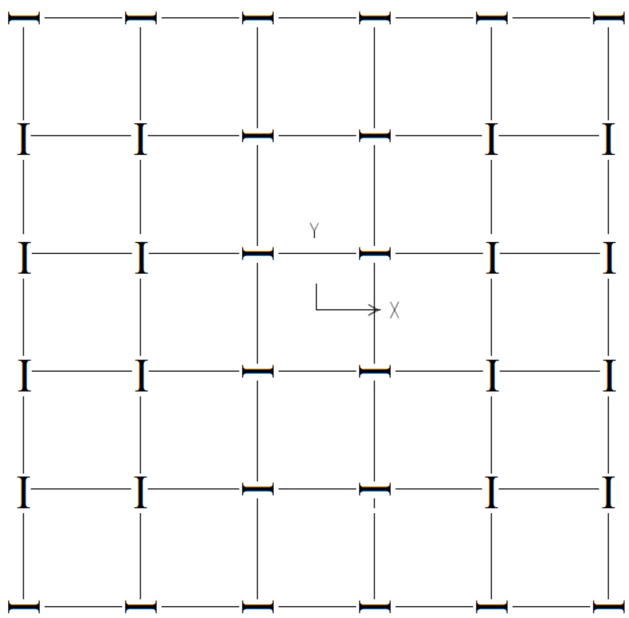
Member Group	Group Name	Member Group	Group Name
1	1st 2nd & 3rd floor corner columns	18	10th floor columns in y-z direction
2	1st 2nd & 3rd floor outer columns in x-z direction	19	1st 2nd & 3rd floor inner columns
3	1st 2nd & 3rd floor outer columns in y-z direction	20	4th &5th floor inner columns
4	1st 2nd & 3rd floor inner corner columns	21	6th &7th floor inner columns
5	4th &5th floor corner columns	22	8th &9th floor inner columns
6	6th &7th floor corner columns	23	10th floor inner columns
7	8th &9th floor corner columns	24	1st 2nd & 3rd floor outer beams

Table 6-15 (continued)

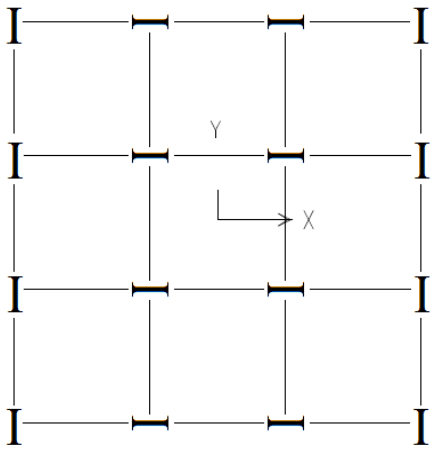
Member Group	Group Name	Member Group	Group Name
8	10th floor corner columns	25	1st 2nd & 3rd floor inner beams
9	1st 2nd & 3rd floor inner columns in x-z direction	26	4th &5th floor outer beams
10	4th &5th floor columns in x-z direction	27	6th &7th floor outer beams
11	6th &7th floor columns in x-z direction	28	8th &9th floor outer beams
12	8th &9th floor columns in x-z direction	29	10th floor outer beams
13	10th floor inner columns in x-z direction	30	4th &5th floor inner beams
14	1st 2nd & 3rd floor inner columns in y-z direction	31	6th &7th floor inner beams
15	4th &5th floor columns in y-z direction	32	8th &9th floor inner beams
16	6th &7th floor columns in y-z direction	33	10th floor inner beams
17	8th &9th floor columns in y-z direction		



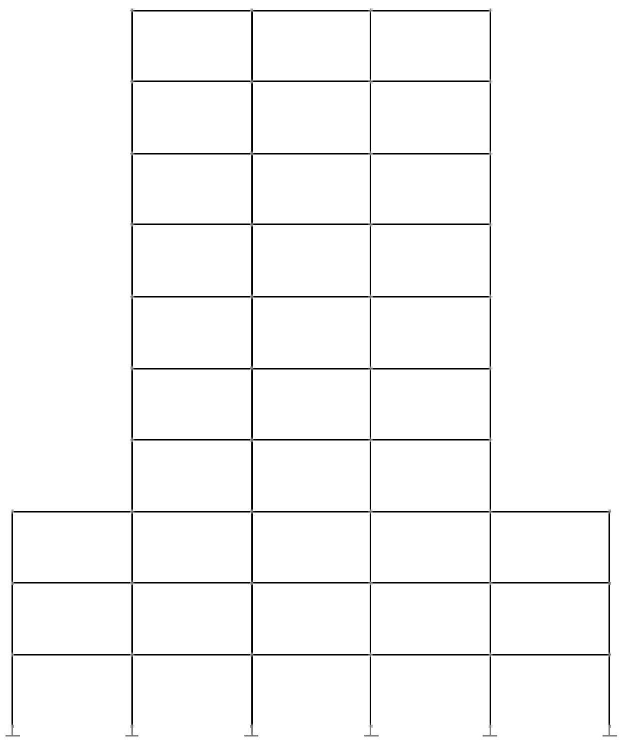
(a) 3D view



(b) Plan view of 5x5 bay

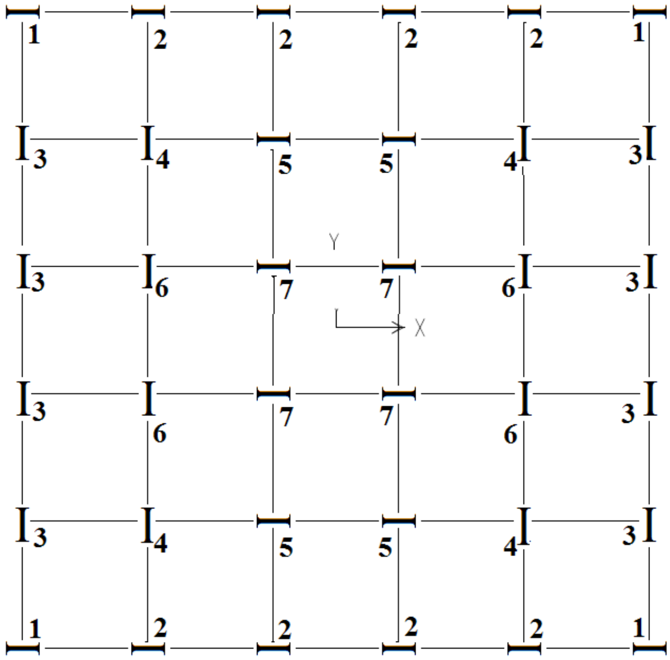


(c) Plan view of 3x3 bay

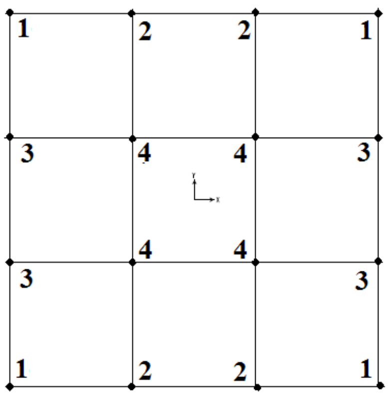


(d) Side view

Figure 6.11 568 member steel frame



(a)Part 1



(b) Part 2

Figure 6.12 Grouping of members of 568-member steel frame in plan level

Table 6-16 Gravity loading on the beams of 568 member steel frame

Beam Type	Uniformly Distributed Load	
	Outer Beams	Inner Beams
	kN/m (lb/ft)	kN/m (lb/ft)
Roof Beams	7.36 (505.87)	14.72 (1011.74)
Floor Beams	10.72 (734.2)	21.43 (1468.4)

(b) 20ft span

Table 6-17 Wind Loading Values under Wind Speed of 105mph(in kN/m,lb/ft).

Floor	z	Windward	Leeward
	(ft)	kN/m (lb/ft)	kN/m (lb/ft)
1	12	1,64 (112,5)	1,86 (127,4)
2	24	1,88 (128,7)	1,86 (127,4)
3	36	2,11 (144,5)	1,86 (127,4)
4	48	2,29 (156,9)	1,86 (127,4)
5	60	2,44 (167,2)	1,86 (127,4)
6	72	2,57 (176,1)	1,86 (127,4)
7	84	2,69 (184,1)	1,86 (127,4)
8	96	2,79 (191,2)	1,86 (127,4)
9	108	2,89 (197,8)	1,86 (127,4)
10	120	1,49 (101,9)	0,93 (63,69)

Table 6-18 Torsional Moment Values (in kNm, lbft).

Floor	z	e_x, e_y	M_T for Case 2	M_T for Case 4
	(ft)	(ft)	kNm (lbft)	kNm (lbft)
1	12	15	365,89 (269870,2)	549,33 (405165,1)
2	24	15	390,56 (288061,2)	586,36 (432475,8)
3	36	15	414,67 (305841,5)	622,55 (459170,1)

Table 6-18 (continued)

Floor	z	e_x, e_y	M_T for Case 2	M_T for Case 4
	(ft)	(ft)	kNm (lbft)	kNm (lbft)
4	48	9	156,08 (115115,8)	234,32 (172827,2)
5	60	9	161,75 (119298)	242,84 (179106)
6	72	9	166,66 (122918,6)	250,21 (184541,8)
7	84	9	171,01 (126130,5)	256,74 (189363,9)
8	96	9	174,94 (129029,4)	262,64 (193716,1)
9	108	9	178,53 (131679,8)	268,04 (197695,3)
10	120	9	90,92 (67063,759)	136,51 (100685)

6.6.2 Test Results

It is mentioned that the structure is analyzed for two different loading conditions. The results of the two cases are presented in the following sections.

6.6.2.1 The Case for Two Load Combinations

The results of the eight runs are tabulated in Table 6-19 in terms of the minimum weight and volume of the frame attained in each run. The design history curves obtained in these runs are plotted in Figure 6.13. It is seen from Table 6-19 or Figure 6.13 that the best design weight of 433,021.0lb (196,417.04kg) is obtained when the frame is sized for minimum weight under initial choice of column orientations. When column orientations are permitted to vary in the optimization process, the best design weight of the frame has been improved to 415,308.52lb (188,382.71kg). The sectional designations attained for member groups in both design cases are compared in Table 6-20. The optimal layout of the column orientation achieved in the latter case is shown in Figure 6.14. It follows that the optimal layout of column orientations can lead to a reduction as much as 4.1 % in the frame weight.

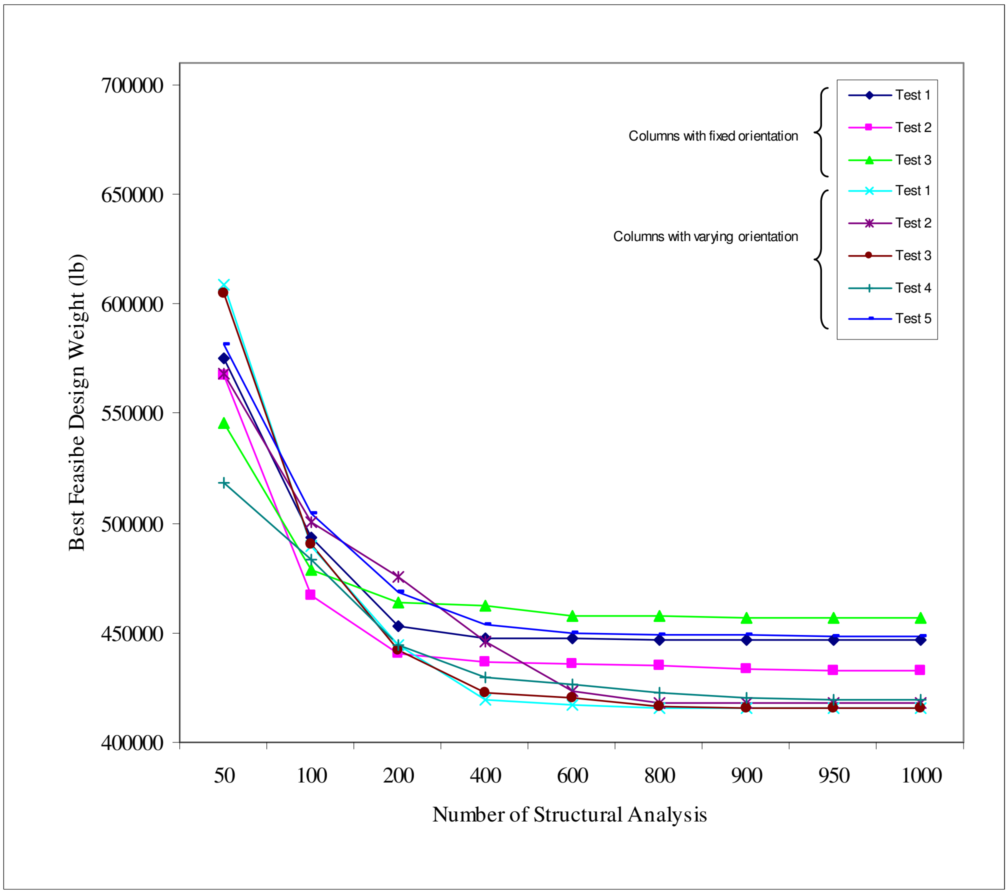


Figure 6.13 Design history graph of 568 member steel frame

Table 6-19 The minimum design weights and volumes obtained for 568-member frame in eight runs.

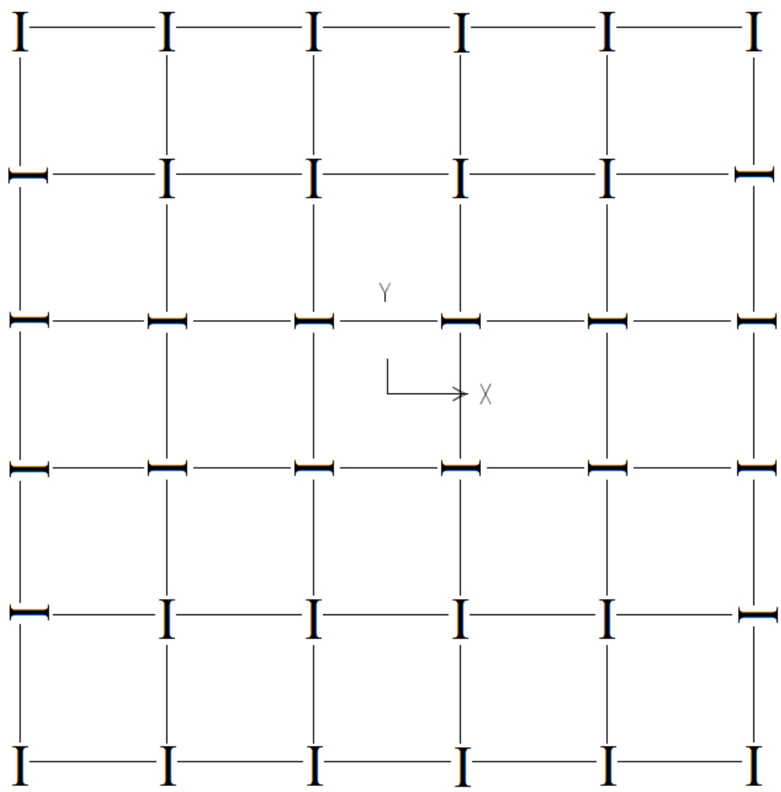
	Columns with fixed orientations			Columns with varying orientations				
	Test1	Test2	Test3	Test1	Test2	Test3	Test4	Test5
Total Weight	446910,9	433021,0	457038,57	415936,1	418030,7	415308,52	419646,1	448392,6
Total Volume	1579190,4	1530109,4	1614977,3	1469738,9	1477140,4	1467521,3	1482848,6	1584426,2

Table 6-20 A comparison of best designs obtained for 568-member steel frame.

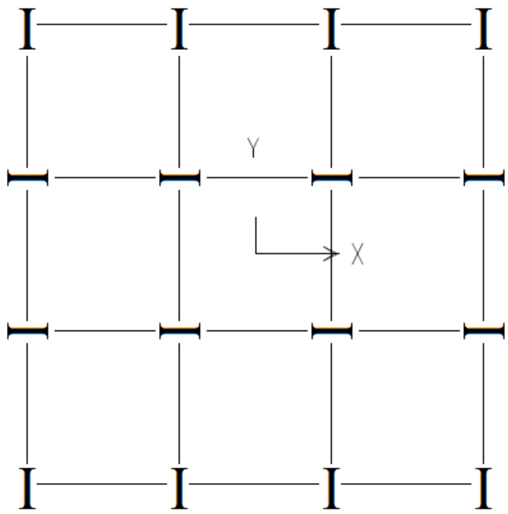
Group Number	Best design with initial column orientations		Best design with varying column orientations	
	Ready Section	Area (cm^2) (in^2)	Ready Section	Area (cm^2) (in^2)
1	W10X49	92,9 (14,4)	W14X48	90,97 (14,1)
2	W8X58	110,32 (17,1)	W8X40	75,48 (11,7)
3	W21X68	129,03 (20)	W14X43	81,29 (12,6)
4	W12X87	165,16 (25,6)	W18X97	183,87 (28,5)
5	W12X58	109,68 (17)	W10X60	113,55 (17,6)
6	W12X53	100,64 (15,6)	W10X49	92,9 (14,4)
7	W10X39	74,19 (11,5)	W8X40	75,48 (11,7)
8	W14X22	41,871 (6,49)	W10X26	49,097 (7,61)
9	W27X146	276,77 (42,9)	W27X146	276,77 (42,9)
10	W14X99	187,74 (29,1)	W14X90	170,97 (26,5)
11	W14X90	170,97 (26,5)	W16X67	127,1 (19,7)
12	W10X60	113,55 (17,6)	W12X53	100,64 (15,6)
13	W10X39	74,19 (11,5)	W10X30	57,032 (8,84)
14	W12X106	201,29 (31,2)	W14X120	227,74 (35,3)
15	W14X90	170,97 (26,5)	W14X90	170,97 (26,5)
16	W12X79	149,68 (23,2)	W14X90	170,97 (26,5)
17	W14X61	115,48 (17,9)	W10X54	101,94 (15,8)
18	W8X28	53,226 (8,25)	W12X26	49,355 (7,65)
19	W40X215	408,39 (63,3)	W40X192	364,52 (56,5)
20	W14X176	334,19 (51,8)	W14X159	301,29 (46,7)
21	W14X109	206,45 (32)	W14X109	206,45 (32)
22	W12X65	123,23 (19,1)	W12X79	149,68 (23,2)
23	W12X26	49,355 (7,65)	W8X24	45,677 (7,08)
24	W14X22	41,871 (6,49)	W12X19	35,935 (5,57)
25	W18X35	66,45 (10,3)	W18X35	66,45 (10,3)
26	W16X26	49,548 (7,68)	W16X26	49,548 (7,68)
27	W16X26	49,548 (7,68)	W14X22	41,871 (6,49)

Table 6-20 (continued)

Group Number	Best design with initial column orientations		Best design with initial column orientations	
	Ready Section	Area (cm2) (in2)	Ready Section	Area (cm2) (in2)
28	W16X26	49,548 (7,68)	W16X26	49,548 (7,68)
29	W12X19	35,935 (5,57)	W12X19	35,935 (5,57)
30	W21X44	83,87 (13)	W24X55	104,52 (16,2)
31	W18X35	66,45 (10,3)	W18X35	66,45 (10,3)
32	W16X31	58,839 (9,12)	W14X30	57,097 (8,85)
33	W14X22	41,871 (6,49)	W12X22	41,806 (6,48)
Weight	433,021.0lb (196,417.04kg)		415,308.52lb (188,382.71kg)	



(a) Part 1



(a) Part 2

Figure 6.14 Optimal layout column orientations for 568-member steel frame.

6.6.2.2 The Case for Nine Load Combinations

The results of the eight runs are tabulated in Table 6-21 in terms of the minimum weight and volume of the frame attained in each run. The design history curves obtained in these runs are plotted in Figure 6.15. It is seen from Table 6-21or Figure 6.15 that the best design weight of 442,643.3lb (200,781.68kg) is obtained when the frame is sized for minimum weight under initial choice of column orientations. When column orientations are permitted to vary in the optimization process, the best design weight of the frame has been improved to 430,086.8lb (195,086.09kg). The sectional designations attained for member groups in both design cases are compared in Table 6-22. The optimal layout of the column orientation achieved in the latter case is shown in Figure 6.16. It follows that the optimal layout of column orientations can lead to a reduction as much as 3.0 % in the frame weight.

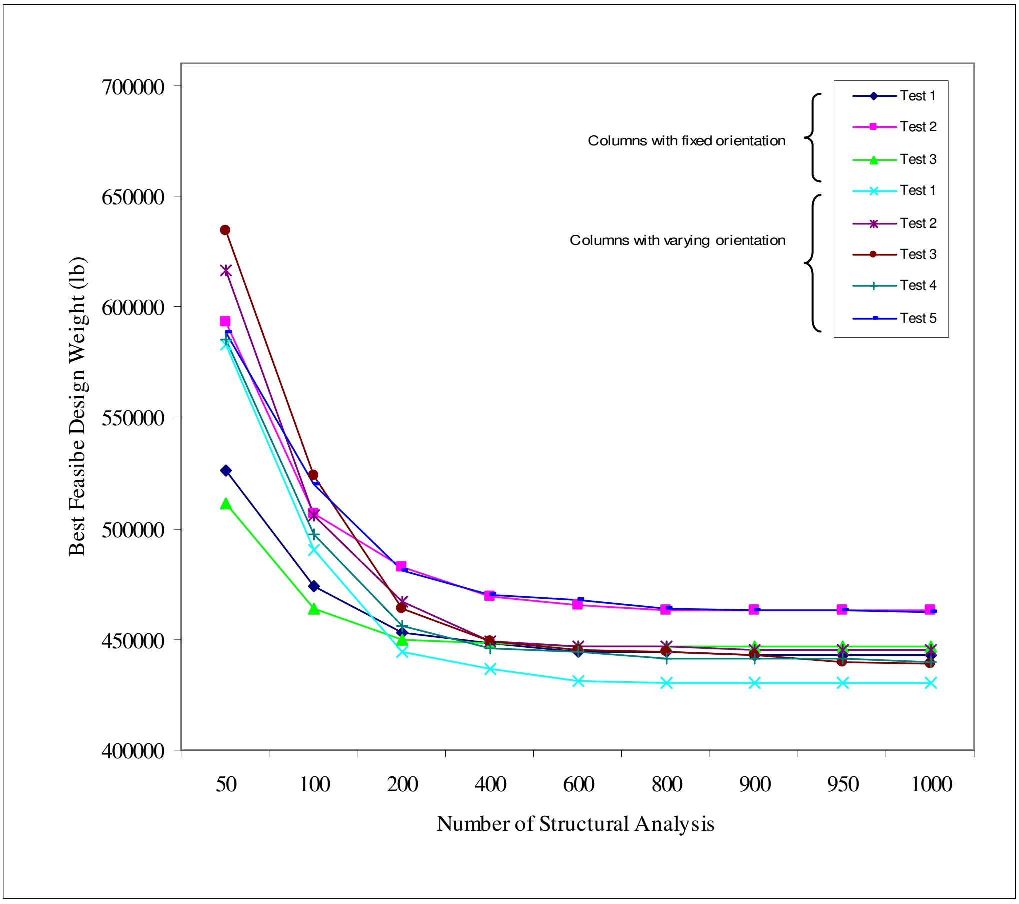


Figure 6.15 Design history graph of 568 member steel frame

Table 6-21 The minimum design weights and volumes obtained for 568-member frame in eight runs.

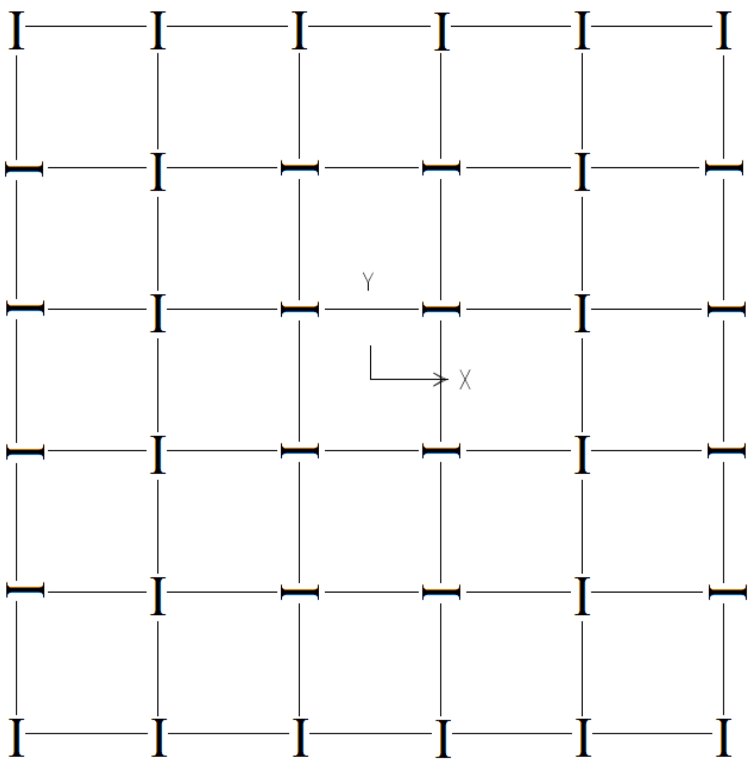
	Columns with fixed orientations			Columns with varying orientations				
	Test1	Test2	Test3	Test1	Test2	Test3	Test4	Test5
Total Weight	442643,3	463366,5	446818,0	430086,8	445404,7	438915,3	439932,5	462253,2
Total Volume	1564110,7	1637337,6	1578862,1	1519741,4	1573868,2	1550937,6	1554531,8	1633403,5

Table 6-22 A comparison of best designs obtained for 568-member steel frame.

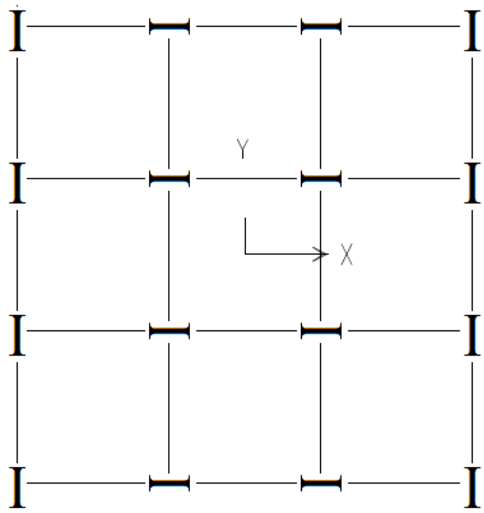
Group Number	Best design with initial column orientations		Best design with varying column orientations	
	Ready Section	Area (cm^2) (in^2)	Ready Section	Area (cm^2) (in^2)
1	W14X38	72,26 (11,2)	W10X49	92,9 (14,4)
2	W21X73	138,71 (21,5)	W10X49	92,9 (14,4)
3	W21X68	129,03 (20)	W10X49	92,9 (14,4)
4	W14X90	170,97 (26,5)	W14X90	170,97 (26,5)
5	W14X90	170,97 (26,5)	W12X72	136,13 (21,1)
6	W12X58	109,68 (17)	W12X53	100,64 (15,6)
7	W12X45	85,16 (13,2)	W14X48	90,97 (14,1)
8	W12X30	56,71 (8,79)	W10X26	49,097 (7,61)
9	W14X109	206,45 (32)	W14X120	227,74 (35,3)
10	W21X111	210,97 (32,7)	W12X106	201,29 (31,2)
11	W12X87	165,16 (25,6)	W14X90	170,97 (26,5)
12	W12X65	123,23 (19,1)	W12X65	123,23 (19,1)
13	W14X38	72,26 (11,2)	W12X30	56,71 (8,79)
14	W36X150	285,16 (44,2)	W36X160	303,23 (47)
15	W14X99	187,74 (29,1)	W14X99	187,74 (29,1)
16	W14X90	170,97 (26,5)	W24X104	197,42 (30,6)
17	W12X65	123,23 (19,1)	W12X65	123,23 (19,1)
18	W14X38	72,26 (11,2)	W12X30	56,71 (8,79)
19	W33X201	381,29 (59,1)	W40X215	408,39 (63,3)
20	W14X145	275,48 (42,7)	W14X145	275,48 (42,7)
21	W14X109	206,45 (32)	W14X99	187,74 (29,1)
22	W12X72	136,13 (21,1)	W12X72	136,13 (21,1)
23	W8X24	45,677 (7,08)	W12X26	49,355 (7,65)
24	W16X26	49,548 (7,68)	W12X19	35,935 (5,57)
25	W18X35	66,45 (10,3)	W18X35	66,45 (10,3)
26	W18X35	66,45 (10,3)	W21X44	83,87 (13)
27	W16X26	49,548 (7,68)	W14X22	41,871 (6,49)

Table 6-22 (continued)

Group Number	Best design with initial column orientations		Best design with initial column orientations	
	Ready Section	Area (cm2) (in2)	Ready Section	Area (cm2) (in2)
28	W12X19	35,935 (5,57)	W12X22	41,806 (6,48)
29	W12X19	35,935 (5,57)	W12X19	35,935 (5,57)
30	W18X35	66,45 (10,3)	W18X35	66,45 (10,3)
31	W18X35	66,45 (10,3)	W18X35	66,45 (10,3)
32	W16X31	58,839 (9,12)	W14X30	57,097 (8,85)
33	W14X22	41,871 (6,49)	W16X26	49,548 (7,68)
Weight	442,643.3lb (200,781.68kg)		430,086.8lb (195,086.09kg)	



(a) Part 1



(a) Part 2

Figure 6.16 Optimal layout column orientations for 568-member steel frame.

6.7 DESIGN EXAMPLE 3: 1230 MEMBER STEEL FRAME

6.7.1 Structural System

The third design example is a 10-story space steel frame consisting of 445 joints and 1230 members. Figure 6.17 shows 3D, elevation and plan views of this structure. The lateral stability of the frame is provided by a combination of exterior and interior rigid (moment-resisting) frames with a core bracing system. The bold lines in Figure 6.18 indicate moment-resisting frameworks that consist of beam and columns that are rigidly connected to each other. The area enclosed by dash lines in Figure 6.18 is the braced core of the structural system, where the dash lines indicate the bracing members. The braced core consists of pin-connections only and is stiffened with K-type bracing in x direction and X-type bracing in y-direction. The braced core is connected to the rigid frameworks with pin-jointed girders. Story heights are 12ft and bays are 15ft long in each of the two directions. All the columns are connected to the foundation with fixed connections.

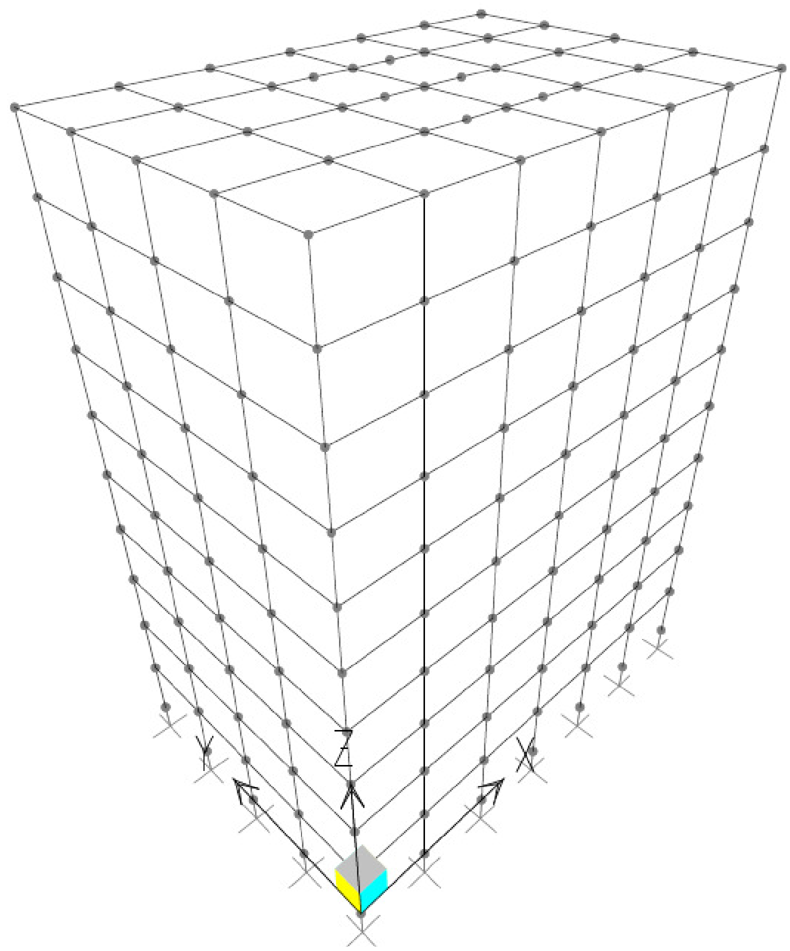
To satisfy practical fabrication requirements, 1230 members of the frame are collected under 50 member size groups; 35 column size groups, 10 beam size groups and 5 bracing size groups. The member groups are clearly tabulated in Table 6-23. The grouping of the members is performed both in plan and elevation levels. A plan level grouping of columns is shown in Figure 6.19. On the other hand, beams are collected under two groups as outer and inner beams in plan level grouping. In elevation level grouping, the member groups are to have the same sections in every two stories.

The orientation variables are assigned, considering the practical requirement that all the successive columns along the frame height must have the same orientation. Hence, each column group in Figure 6.19 is associated with an orientation variable, resulting in seven orientation variables in all.

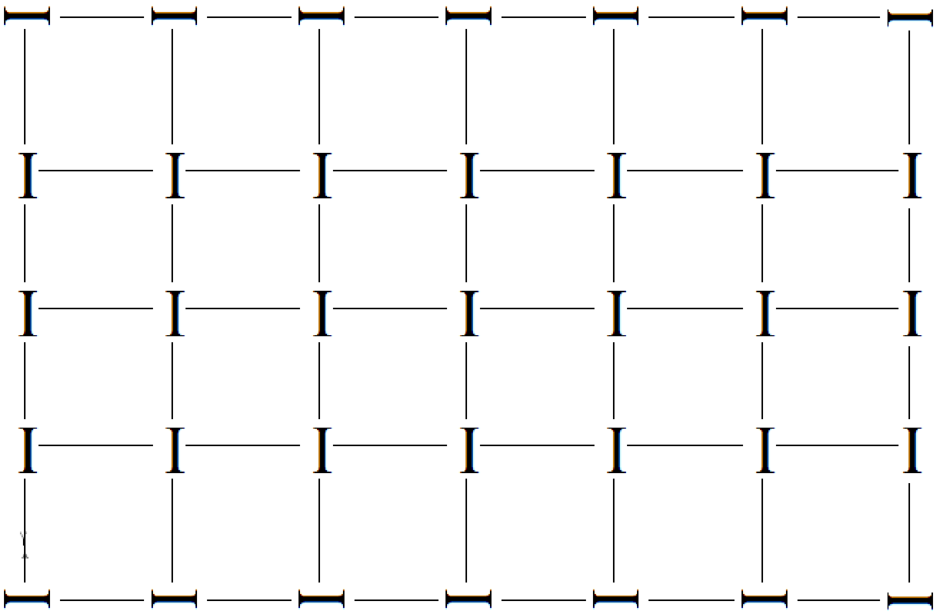
The frame is subjected to two loading conditions of combined gravity and wind forces. Load cases and combinations are determined according to the ASCE 7-05 [43], as explained in Section 6.2. The 1st and the 2nd combinations that are tabulated in Table 6-2 are taken into consideration for this design example. The resulting gravity loading on the beams of roof and floors is tabulated in Table 6-24 and the wind loadings on windward and leeward faces of the frame calculated based on a basic wind speed of 105mph (46.94 m/s) is presented in Table6-25.

The wide-flange (W) profile list consisting of 297 ready sections is used to size column members, while beams and diagonals are selected from discrete sets of 171 and 147 economical sections selected from wide-flange profile list based on area and inertia properties in the former, and on area and radii of gyration properties in the latter. Provisions of ASD-AISC [39] are taken into consideration for the stress, stability, and geometric constraints. Displacements of all the joints in x and y directions are limited to 9.15cm (3.6in), and the upper limit of inter story drifts is set to 0.91cm (0.36in).

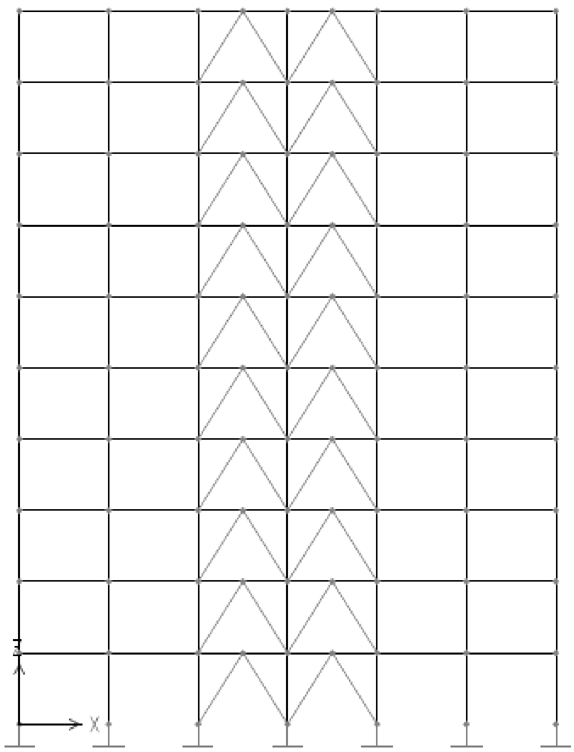
Two design cases are studied. First, the structure is designed for minimum weight considering the size design variables only, where initial orientations of the column members are kept unchanged. Second, size and orientation variables are employed together to minimize the weight of the frame.



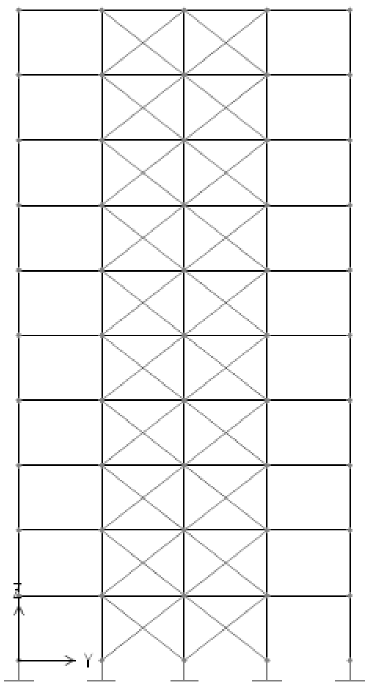
(a) 3D view



(c)Plan view



(c) Side view



(d) Side view

Figure 6.17 1230-member steel frame

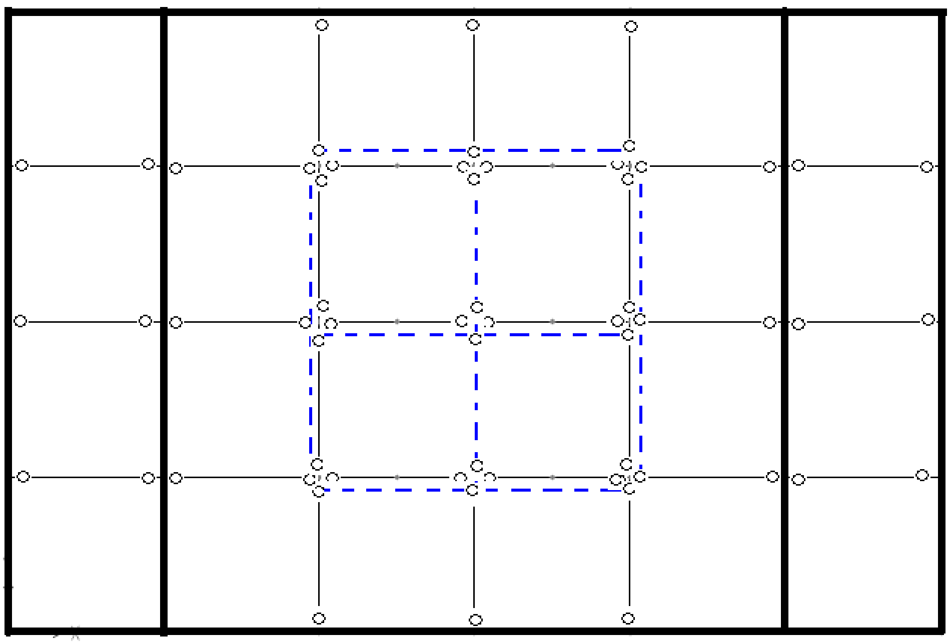


Figure 6.18 Vertical bracings and moment releases of 1230 member steel frame

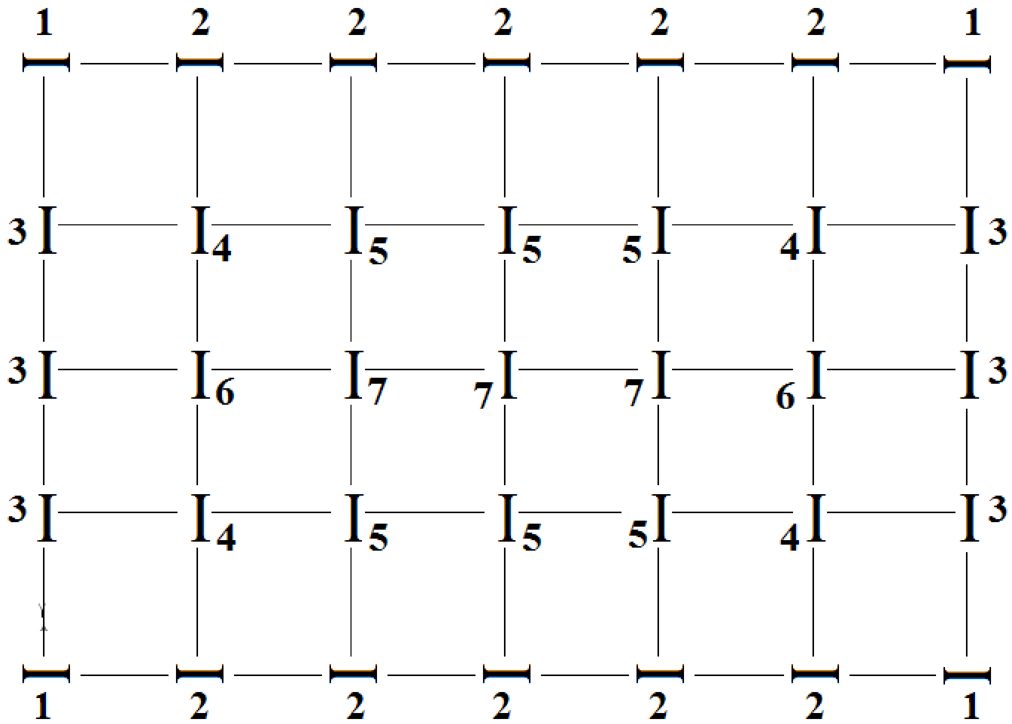


Figure 6.19 Grouping of members of 1230-member steel frame in plan view

Table 6-23 Member grouping for 1230 member steel frame.

Member Group	Group Name	Member Group	Group Name
1	Corner Columns of 1^{st} & 2^{nd} floor	26	Inner Y-Z Outer Columns of 1^{st} & 2^{nd} floor
2	Corner Columns of 3^{rd} & 4^{th}floor	27	Inner Y-Z Outer Columns of 3^{rd} & 4^{th}floor
3	Corner Columns of 5^{th} &6^{th} floor	28	Inner Y-Z Outer Columns of 5^{th} &6^{th} floor
4	Corner Columns of 7^{th} & 8^{th} floor	29	Inner Y-Z Outer Columns of 7^{th} & 8^{th} floor
5	Corner Columns of 9^{th} & 10^{th} floor	30	Inner Y-Z Outer Columns of 9^{th} & 10^{th} floor
6	X-Z Outer Columns of 1^{st} & 2^{nd} floor	31	Inner Columns of 1^{st} & 2^{nd} floor

Table 6-23 (continued)

Member Group	Group Name	Member Group	Group Name
7	X-Z Outer Columns of 3^{rd} & 4^{th}floor	32	Inner Columns of 3^{rd} & 4^{th}floor
8	X-Z Outer Columns of 5^{th} &6^{th} floor	33	Inner Columns of 5^{th} &6^{th} floor
9	X-Z Outer Columns of 7^{th} & 8^{th} floor	34	Inner Columns of 7^{th} & 8^{th} floor
10	X-Z Outer Columns of 9^{th} & 10^{th} floor	35	Inner Columns of 9^{th} & 10^{th} floor
11	Y-Z Outer Columns of 1^{st} & 2^{nd} floor	36	Outer beams of 1^{st} &2^{nd} floor
12	Y-Z Outer Columns of 3^{rd} & 4^{th}floor	37	Outer beams of 3^{rd} & 4^{th}floor
13	Y-Z Outer Columns of 5^{th} &6^{th} floor	38	Outer beams of 5^{th} &6^{th} floor
14	Y-Z Outer Columns of 7^{th} & 8^{th} floor	39	Outer beams of 7^{th} & 8^{th} floor
15	Y-Z Outer Columns of 9^{th} & 10^{th} floor	40	Outer beams of 9^{th} & 10^{th} floor
16	Inner Corner Columns of 1^{st} & 2^{nd} floor	41	Inner beams of 1^{st} &2^{nd} floor
17	Inner Corner Columns of 3^{rd} & 4^{th}floor	42	Inner beams of 3^{rd} & 4^{th}floor
18	Inner Corner Columns of 5^{th} &6^{th} floor	43	Inner beams of 5^{th} &6^{th} floor
19	Inner Corner Columns of 7^{th} & 8^{th} floor	44	Inner beams of 7^{th} & 8^{th} floor
20	Inner Corner Columns of 9^{th} & 10^{th} floor	45	Inner beams of 9^{th} & 10^{th} floor
21	Inner X-Z Outer Columns of 1^{st} & 2^{nd} floor	46	Bracings of 1^{st} &2^{nd} floor
22	Inner X-Z Outer Columns of 3^{rd} & 4^{th}floor	47	Bracings of 3^{rd} & 4^{th}floor

Table 6-23 (continued)

Member Group	Group Name	Member Group	Group Name
23	Inner X-Z Outer Columns of 5th &6th floor	48	Bracings of 5th &6th floor
24	Inner X-Z Outer Columns of 7th & 8th floor	49	Bracings of 7th & 8th floor
25	Inner X-Z Outer Columns of 9th & 10th floor	50	Bracings of 9th & 10th floor

Table 6-24 Gravity loading on the beams of 1230 member steel frame.

Beam Type	Uniformly Distributed Load	
	Outer Beams	Inner Beams
	kN/m(lb/ft)	kN/m(lb/ft)
Roof Beams	5.54 (379.4)	11.08 (758.8)
Floor Beams	8.04 (550.65)	16,08 (1101.3)

(a)15ft span

Table6-25 Wind Loading Values under Wind Speed of 105mph (in kN/m,lb/ft).

Floor	z	Windward	Leeward
	(ft)	kN/m (lb/ft)	kN/m (lb/ft)
1	12	1,64 (112,5)	1,86 (127,4)
2	24	1,88 (128,7)	1,86 (127,4)
3	36	2,11 (144,5)	1,86 (127,4)
4	48	2,29 (156,9)	1,86 (127,4)
5	60	2,44 (167,2)	1,86 (127,4)
6	72	2,57 (176,1)	1,86 (127,4)
7	84	2,69 (184,1)	1,86 (127,4)
8	96	2,79 (191,2)	1,86 (127,4)
9	108	2,89 (197,8)	1,86 (127,4)
10	120	1,49 (101,9)	0,93 (63,69)

6.7.2 Test Results

The results of the eight runs are tabulated in Table 6-26 in terms of the minimum weight and volume of the frame attained in each run. The design history curves obtained in these runs are plotted in Figure 6.20. It is seen from Table 6-26 or Figure 6.20 that the best design weight of 470,970.18lb (213,632.07 kg) is obtained when the frame is sized for minimum weight under initial choice of column orientations. When column orientations are permitted to vary in the optimization process, the best design weight of the frame has been improved to 442,626.99 lb (200,775.6 kg). The sectional designations attained for member groups in both design cases are compared in Table 6-27. The optimal layout of the column orientation achieved in the latter case is shown in Figure 6.21. It follows that the optimal layout of column orientations can lead to a reduction as much as 6.0 % in the frame weight.

Table 6-26 The minimum design weights and volumes obtained for 1230-member frame in eight runs.

	Columns with fixed orientations			Columns with varying orientations				
	Test1	Test2	Test3	Test1	Test2	Test3	Test4	Test5
Total Weight	501139,5	475927,0	470970,2	452929,9	446046,8	456639,8	442627,0	443261,7
Total Volume	1770810,9	1681720,8	1664205,6	1600459,1	1576137	1613568,1	1564053	1566295,6

Table 6-27 A comparison of best designs obtained for 1230-member steel frame.

Group Number	Best design with initial column orientations		Best design with varying column orientations	
	Ready Section	Area (in^2) (cm^2)	Ready Section	Area (in^2) (cm^2)
1	W12X30	56,71 (8,79)	W14X30	57,097 (8,85)
2	W10X39	74,19 (11,5)	W8X35	66,45 (10,3)
3	W10X33	62,645 (9,71)	W6X25	47,355 (7,34)
4	W8X28	53,226 (8,25)	W10X22	41,871 (6,49)
5	W12X26	49,355 (7,65)	W8X24	45,677 (7,08)
6	W8X48	90,97 (14,1)	W16X50	94,84 (14,7)
7	W10X49	92,9 (14,4)	W16X36	68,39 (10,6)
8	W10X39	74,19 (11,5)	W8X28	53,226 (8,25)
9	W10X26	49,097 (7,61)	W6X25	47,355 (7,34)
10	W8X24	45,677 (7,08)	W8X24	45,677 (7,08)
11	W16X45	85,81 (13,3)	W16X45	85,81 (13,3)
12	W16X40	76,13 (11,8)	W10X33	62,645 (9,71)
13	W14X30	57,097 (8,85)	W18X35	66,45 (10,3)
14	W10X22	41,871 (6,49)	W10X22	41,871 (6,49)
15	W8X24	45,677 (7,08)	W8X24	45,677 (7,08)
16	W36X160	303,23 (47)	W30X90	170,32 (26,4)
17	W24X84	159,35 (24,7)	W18X65	123,23 (19,1)
18	W24X62	117,42 (18,2)	W21X57	107,74 (16,7)
19	W10X33	62,645 (9,71)	W10X33	62,645 (9,71)
20	W8X24	45,677 (7,08)	W12X30	56,71 (8,79)
21	W33X118	223,87 (34,7)	W18X76	143,87 (22,3)
22	W12X72	136,13 (21,1)	W10X54	101,94 (15,8)
23	W8X48	90,97 (14,1)	W8X40	75,48 (11,7)
24	W12X26	49,355 (7,65)	W8X28	53,226 (8,25)
25	W8X24	45,677 (7,08)	W8X24	45,677 (7,08)
26	W24X84	159,35 (24,7)	W18X76	143,87 (22,3)
27	W24X68	129,68 (20,1)	W12X72	136,13 (21,1)

Table 6-27 (continued)

Group Number	Best design with initial column orientations		Best design with varying column orientations	
	Ready Section	Area (in^2) (cm^2)	Ready Section	Area (in^2) (cm^2)
28	W18X50	94,84 (14,7)	W16X57	108,39 (16,8)
29	W18X40	76,13 (11,8)	W12X35	66,45 (10,3)
30	W8X24	45,677 (7,08)	W8X28	53,226 (8,25)
31	W36X160	303,23 (47)	W36X160	303,23 (47)
32	W21X83	156,77 (24,3)	W12X72	136,13 (21,1)
33	W16X57	108,39 (16,8)	W8X67	127,1 (19,7)
34	W16X36	68,39 (10,6)	W10X33	62,645 (9,71)
35	W8X24	45,677 (7,08)	W8X28	53,226 (8,25)
36	W16X26	49,548 (7,68)	W14X22	41,871 (6,49)
37	W8X18	33,935 (5,26)	W10X22	41,871 (6,49)
38	W8X18	33,935 (5,26)	W8X18	33,935 (5,26)
39	W8X18	33,935 (5,26)	W8X18	33,935 (5,26)
40	W6X15	28,581 (4,43)	W6X15	28,581 (4,43)
41	W12X26	49,355 (7,65)	W12X26	49,355 (7,65)
42	W16X31	58,839 (9,12)	W12X26	49,355 (7,65)
43	W14X30	57,097 (8,85)	W12X26	49,355 (7,65)
44	W16X26	49,548 (7,68)	W10X26	49,097 (7,61)
45	W8X24	45,677 (7,08)	W8X24	45,677 (7,08)
46	W6X15	28,581 (4,43)	W6X20	37,871 (5,87)
47	W6X15	28,581 (4,43)	W6X20	37,871 (5,87)
48	W6X15	28,581 (4,43)	W6X15	28,581 (4,43)
49	W6X15	28,581 (4,43)	W6X15	28,581 (4,43)
50	W6X15	28,581 (4,43)	W6X15	28,581 (4,43)
Weight	213632.07kg (470970.18lb)		200775.6kg (442626.99lb)	

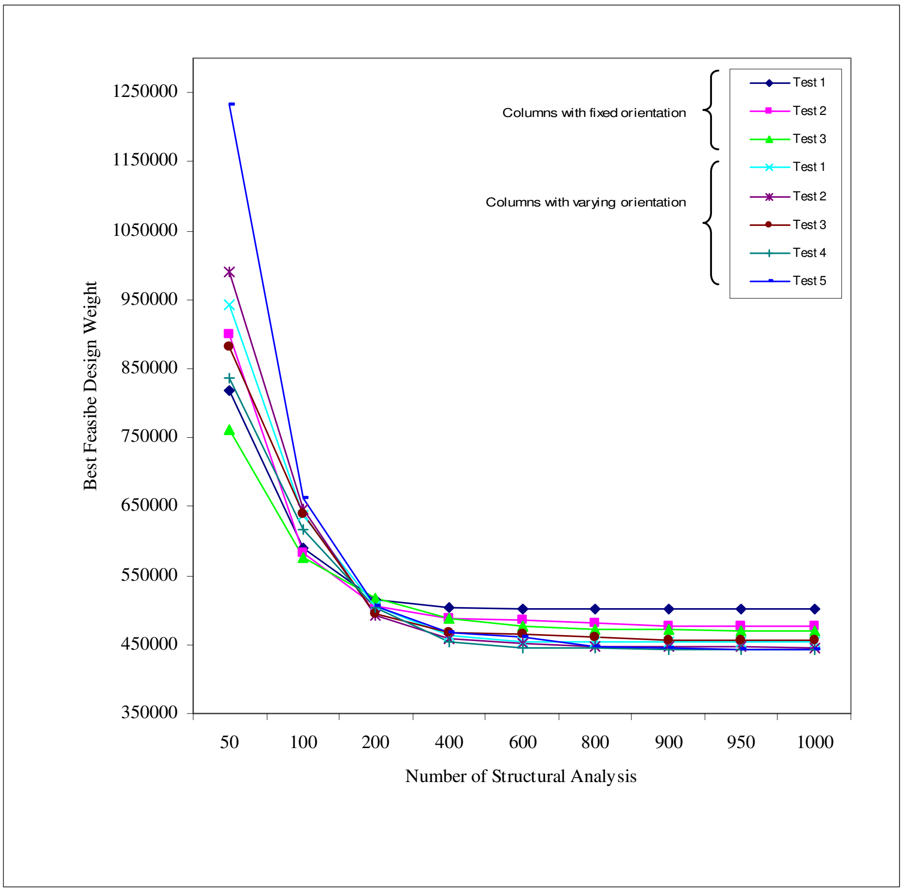


Figure 6.20 Design history graph of 1230 member steel frame

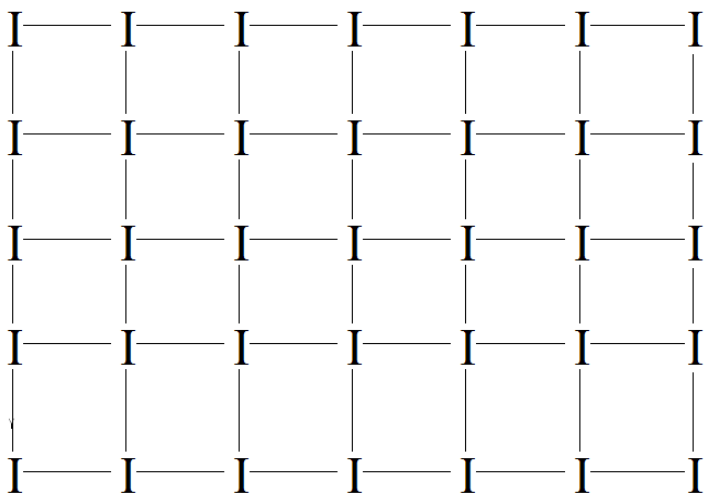


Figure 6.21 Optimal layout column orientation for 1230-member steel frame.

6.8 DESIGN EXAMPLE 4: 3590 MEMBER STEEL FRAME

6.8.1 Structural System

The fourth (last) design example is a braced space steel frame consisting of 1540 joints and 3590 members that are to be built in three adjacent blocks. The 3D, side and plan views of the frame at different story levels are shown in Figure 6.22. The first, second and third blocks consist of 12, 18 and 30 stories with an equal story height of 12ft. The bays are 24ft long in both directions. An economical and effective stiffening of the frame against lateral forces is achieved through exterior diagonal bracing members located on the perimeter of the building as well as on the adjacent sides of the blocks. The diagonal members are also known to participate in transmitting gravity forces. All the columns are connected to the foundation with fixed connections.

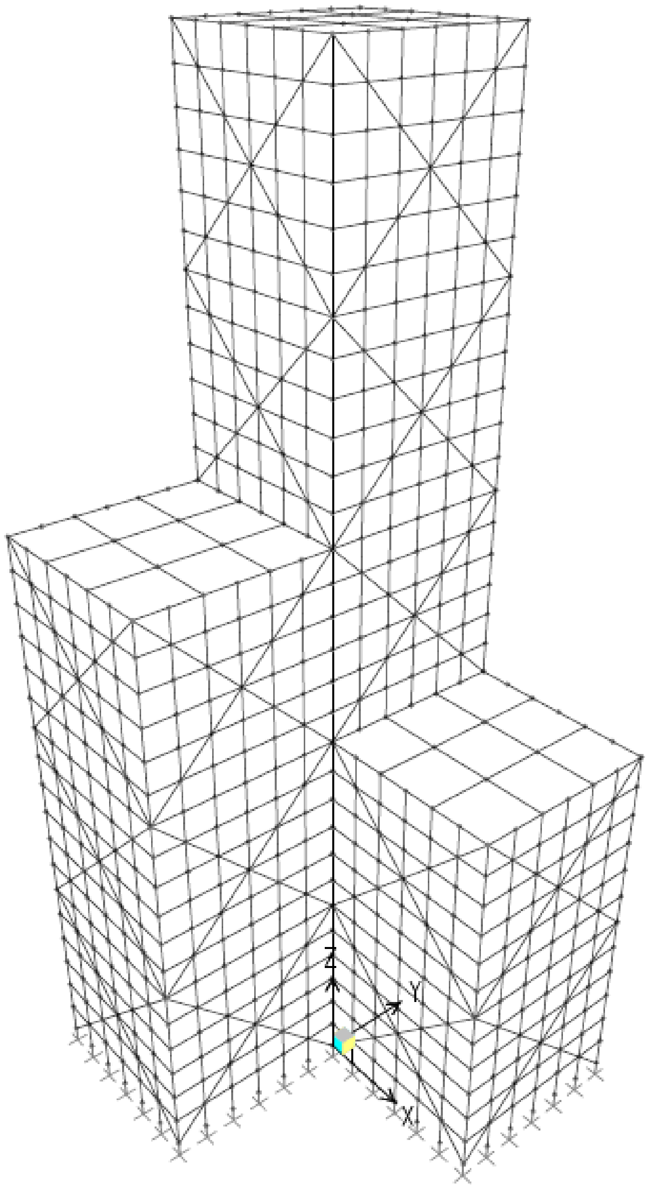
To satisfy practical fabrication requirements, 3590 members of the frame are collected under 129 member size groups; 104 column size groups, 20 beam size groups and 5 bracing size groups. The member groups are clearly tabulated in Table 6-28. The grouping of the members is performed both in plan and elevation levels. A plan level grouping of columns is shown in Figure 6.23. The columns are collected in 16 size groups at each level between 1-12th stories (Figure 6.23a), in 10 groups at each level between 13-18th stories (Figure 6.23b), and in 4 groups at each level between 19-30th stories (Figure 6.23c). On the other hand, beams are collected under two groups as outer and inner beams in plan level grouping. In elevation level grouping, the column size groups are to have the same section over three adjacent stories, as are beam size groups. Bracing members are designed as three-story deep members, and a single bracing size group is specified in every six stories.

The orientation variables are assigned, keeping in mind that all the successive columns along the frame height must have the same orientation. Hence, each column group in Figure 6.23(a) is associated with an orientation variable, resulting in sixteen orientation variables in all.

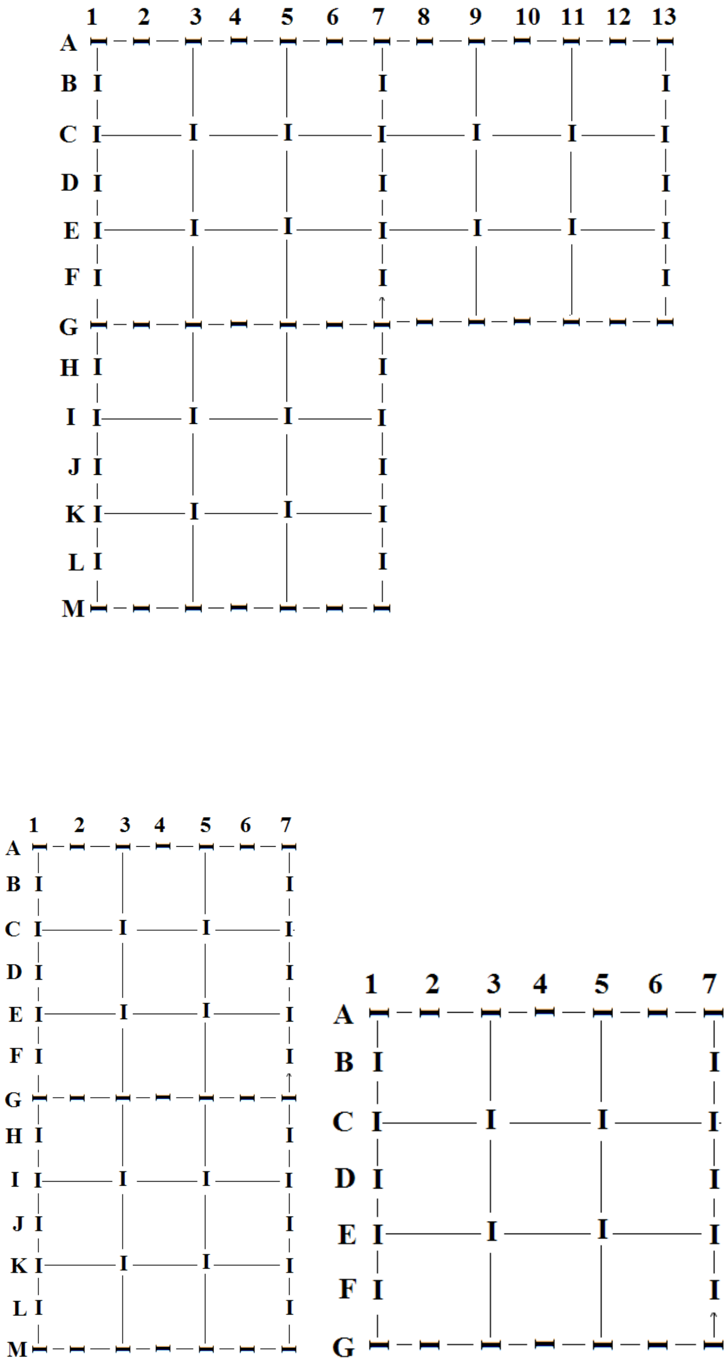
The frame is subjected to two loading conditions of combined gravity and wind forces. Load cases and combinations are determined according to the ASCE 7-05 [43], as explained in Section 6.2. Only the 1st and the 2nd combinations that are tabulated in Table 6-2 are taken into consideration for this design example. The resulting gravity loading on the beams of roof and floors is tabulated in Table 6-29 and the wind loadings on windward and leeward faces of the frame calculated based on a basic wind speed of 105mph (46.94 m/s) is presented in Table 6-30.

The wide-flange (W) profile list consisting of 297 ready sections is used to size column members, while beams and diagonals are selected from discrete sets of 171 and 147 economical sections selected from wide-flange profile list based on area and inertia properties in the former, and on area and radii of gyration properties in the latter. Provisions of ASD-AISC [39] are taken into consideration for the stress, stability, and geometric constraints. Displacements of all the joints in x and y directions are limited to 27.43cm (10.8 in), and the upper limit of inter story drifts is set to 0.91cm (0.36in), which is equal to the story height/400.

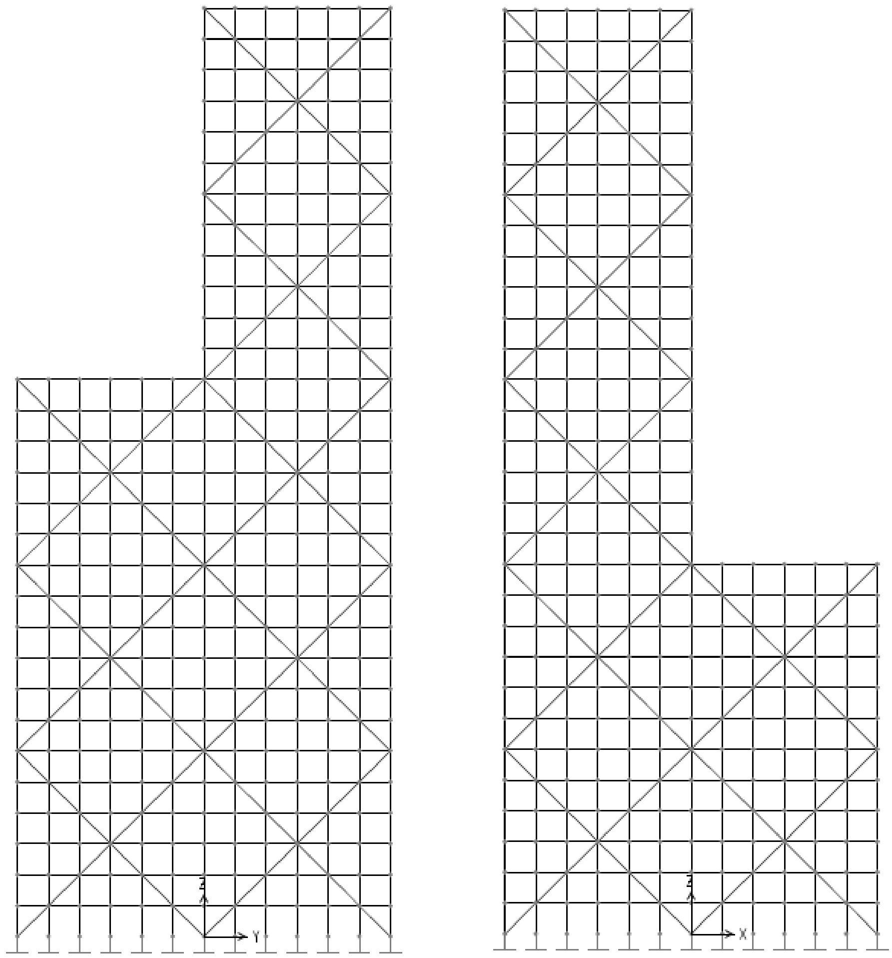
Two design cases are studied. First, the structure is designed for minimum weight considering the size design variables only, where initial orientations of the column members are kept unchanged. Second, size and orientation variables are employed together to minimize the weight of the frame.



(a) 3D view


1 2 3 4 5 6 7 8 9 10 11 12 13
A B C D E F G H I J K L M
1 2 3 4 5 6 7
A B C D E F G H I J K L M
1 2 3 4 5 6 7
A B C D E F G

(b) Plan view



(d) Side view

Figure 6.22 3590 member steel frame

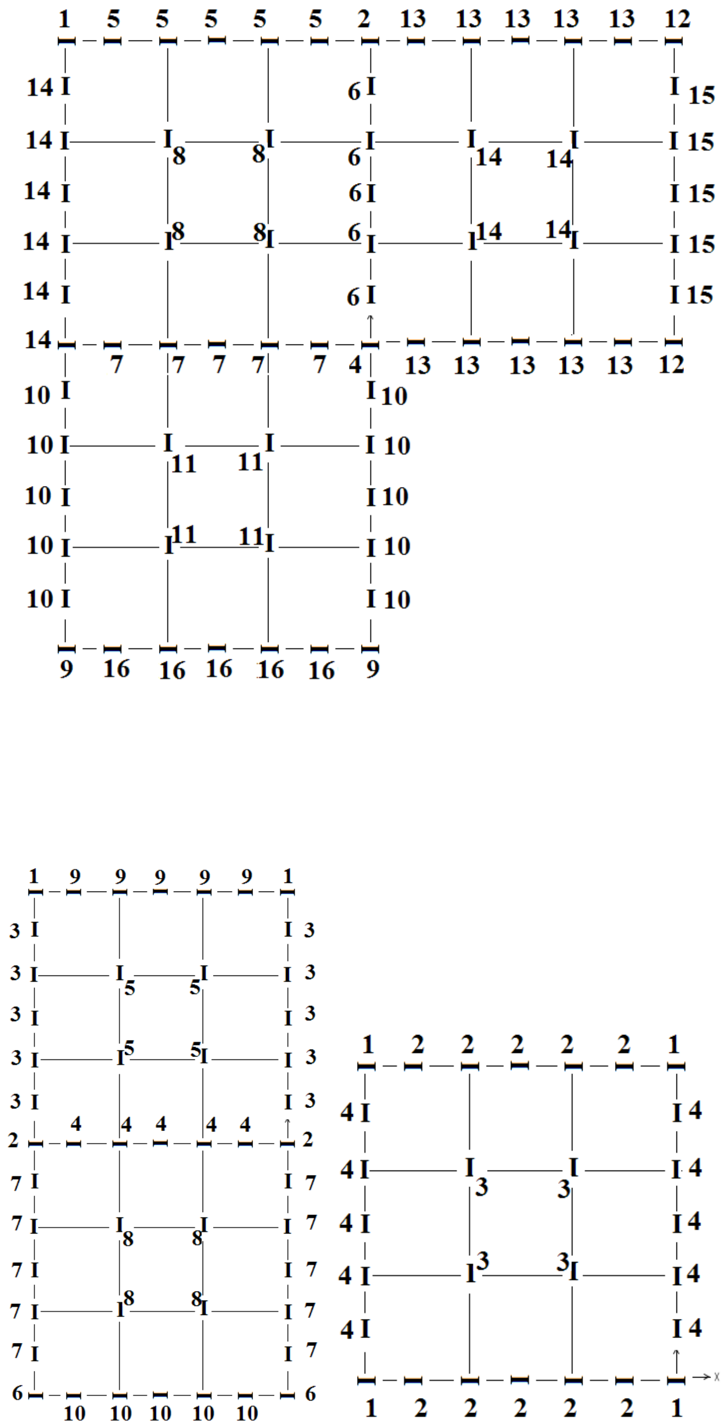


Figure 6.23 Grouping of members of 3590-member steel frame in plan level.

Table 6-28 Member grouping for 3590-member steel frame

Member Group	Group Name	Member Group	Group Name
1st, 2nd and 3rd floors			
1	Corner columns on axis A1	10	Corner columns on axis M1, M7
2	Corner columns on axis A7	11	Outer columns on axis H-L/1, H-L/7
3	Corner columns on axis G1	12	Outer columns on axis M/2-6
4	Corner columns on axis G7	13	Inner columns on axis I/3-4, K/3-4
5	Outer columns on axis B-F/1	14	Corner columns on axis A13 and G13
6	Outer columns on axis A/2-6	15	Outer columns on axis A/8-12, G/8-12
7	Inner columns on axis B-F/7	16	Outer columns on axis B-F/13
8	Inner columns on axis G/2-6	17	Inner columns on axis C/9-11, E/9-11
9	Inner columns on axis C/3-4, E/3-4		
4th, 5th and 6th floors			
18	Corner columns on axis A1	27	Corner columns on axis M1, M7
19	Corner columns on axis A7	28	Outer columns on axis H-L/1, H-L/7
20	Corner columns on axis G1	29	Outer columns on axis M/2-6
21	Corner columns on axis G7	30	Inner columns on axis I/3-4, K/3-4
22	Outer columns on axis B-F/1	31	Corner columns on axis A13 and G13

Table 6-28 (continued)

Member Group	Group Name	Member Group	Group Name
23	Outer columns on axis A/2-6	32	Outer columns on axis A/8-12, G/8-12
24	Inner columns on axis B-F/7	33	Outer columns on axis B-F/13
25	Inner columns on axis G/2-6	34	Inner columns on axis C/9-11, E/9-11
26	Inner columns on axis C/3-4, E/3-4		
7th, 8th and 9th floors			
35	Corner columns on axis A1	44	Corner columns on axis M1, M7
36	Corner columns on axis A7	45	Outer columns on axis H-L/1, H-L/7
37	Corner columns on axis G1	46	Outer columns on axis M/2-6
38	Corner columns on axis G7	47	Inner columns on axis I/3-4, K/3-4
39	Outer columns on axis B-F/1	48	Corner columns on axis A13 and G13
40	Outer columns on axis A/2-6	49	Outer columns on axis A/8-12, G/8-12
41	Inner columns on axis B-F/7	50	Outer columns on axis B-F/13
42	Inner columns on axis G/2-6	51	Inner columns on axis C/9-11, E/9-11
43	Inner columns on axis C/3-4, E/3-4		
10th, 11th and 12th floors			
52	Corner columns on axis A1	61	Corner columns on axis M1, M7
53	Corner columns on axis A7	62	Outer columns on axis H-L/1, H-L/7

Table 6-28 (continued)

Member Group	Group Name	Member Group	Group Name
54	Corner columns on axis G1	63	Outer columns on axis M/2-6
55	Corner columns on axis G7	64	Inner columns on axis I/3-4, K/3-4
56	Outer columns on axis B-F/1	65	Corner columns on axis A13 and G13
57	Outer columns on axis A/2-6	66	Outer columns on axis A/8-12, G/8-12
58	Inner columns on axis B-F/7	67	Outer columns on axis B-F/13
59	Inner columns on axis G/2-6	68	Inner columns on axis C/9-11, E/9-11
60	Inner columns on axis C/3-4, E/3-4		
13th, 14th and 15th floors			
69	Corner columns on axis A1, A7	74	Outer columns on axis M/2-6
70	Corner columns on axis G1, G7	75	Outer columns on axis H-L/1, H-L/7
71	Corner columns on axis M1, M7	76	Inner columns on axis G/2-6
72	Outer columns on axis B-F/1, B-F/7	77	Inner columns on axis C/3-4, E/3-4
73	Outer columns on axis A/2-6	78	Inner columns on axis I/3-4, K/3-4
16th, 17th and 18th floors			
79	Corner columns on axis A1, A7	84	Outer columns on axis M/2-6
80	Corner columns on axis G1, G7	85	Outer columns on axis H-L/1, H-L/7
81	Corner columns on axis M1, M7	86	Inner columns on axis G/2-6

Table 6-28 (continued)

Member Group	Group Name	Member Group	Group Name
82	Outer columns on axis B-F/1, B-F/7	87	Inner columns on axis C/3-4, E/3-4
83	Outer columns on axis A/2-6	88	Inner columns on axis I/3-4, K/3-4
19^{th}, 20^{th} and 21^{st} floors			
89	Corner columns on axis A1, A7, G1 and G7	91	Outer columns on axis B-F/1, B-F/7
90	Outer columns on axis A/2-6, G/2-6	92	Inner columns on axis C/3-4, E/3-4
22^{nd}, 23^{rd} and 24^{th} floors			
93	Corner columns on axis A1, A7, G1 and G7	95	Outer columns on axis B-F/1, B-F/7
94	Outer columns on axis A/2-6, G/2-6	96	Inner columns on axis C/3-4, E/3-4
25^{th}, 26^{th} and 27^{th} floors			
97	Corner columns on axis A1, A7, G1 and G7	99	Outer columns on axis B-F/1, B-F/7
98	Outer columns on axis A/2-6, G/2-6	100	Inner columns on axis C/3-4, E/3-4
28^{th}, 29^{th} and 30^{th} floors			
101	Corner columns on axis A1, A7, G1 and G7	103	Outer columns on axis B-F/1, B-F/7
102	Outer columns on axis A/2-6, G/2-6	104	Inner columns on axis C/3-4, E/3-4
Beams			
105	Inner beams between 1-3 stories	115	Inner beams between 16-18 stories
106	Outer beams between 1-3 stories	116	Outer beams between 16-18 stories
107	Inner beams between 4-6 stories	117	Inner beams between 19-21 stories

Table 6-28 (continued)

Member Group	Group Name	Member Group	Group Name
108	Outer beams between 4-6 stories	118	Outer beams between 19-21 stories
109	Inner beams between 7-9 stories	119	Inner beams between 22-24 stories
110	Outer beams between 7-9 stories	120	Outer beams between 22-24 stories
111	Inner beams between 10-12 stories	121	Inner beams between 25-27 stories
112	Outer beams between 10-12 stories	122	Outer beams between 25-27 stories
113	Inner beams between 13-15 stories	123	Inner beams between 28-30 stories
114	Outer beams between 13-15 stories	124	Outer beams between 28-30 stories
Bracings			
125	Bracings between 1-6 stories	128	Bracings between 19-24 stories
126	Bracings between 7-12 stories	129	Bracings between 25-30 stories
127	Bracings between 13-18 stories		

Table 6-29 Gravity loading on the beams of 3590 member steel frame.

Beam Type	Uniformly Distributed Load	
	Outer Beams	Inner Beams
	kN/m(lb/ft)	kN/m(lb/ft)
Roof Beams	8.86 (607.04)	17.72 (1214.08)
Floor Beams	12.86 (881.04)	25.72 (1762.08)

(a) 24ft span

Table 6-30 Wind Loading Values under Wind Speed of 105mph (in kN/m,lb/ft).

Floor	z	Windward	Leeward
	(ft)	kN/m (lb/ft)	kN/m (lb/ft)
1	12	1,64 (112,5)	2,55 (174,3)
2	24	1,88 (128,7)	2,55 (174,3)
3	36	2,11 (144,5)	2,55 (174,3)
4	48	2,29 (156,9)	2,55 (174,3)
5	60	2,44 (167,2)	2,55 (174,3)
6	72	2,57 (176,1)	2,55 (174,3)
7	84	2,69 (184,1)	2,55 (174,3)
8	96	2,79 (191,2)	2,55 (174,3)
9	108	2,89 (197,8)	2,55 (174,3)
10	120	2,98 (203,8)	2,55 (174,3)
11	132	3,06 (209,4)	2,55 (174,3)
12	144	3,13 (214,7)	2,55 (174,3)
13	156	3,21 (219,7)	2,55 (174,3)
14	168	3,28 (224,4)	2,55 (174,3)
15	180	3,34 (228,8)	2,55 (174,3)
16	192	3,4 (233,1)	2,55 (174,3)
17	204	3,46 (237,2)	2,55 (174,3)
18	216	3,52 (241,1)	2,55 (174,3)
19	228	3,57 (244,8)	2,55 (174,3)
20	240	3,63 (248,4)	2,55 (174,3)
21	252	3,68 (251,9)	2,55 (174,3)
22	264	3,73 (255,3)	2,55 (174,3)
23	276	3,77 (258,6)	2,55 (174,3)
24	288	3,82 (261,7)	2,55 (174,3)
25	300	3,87 (264,8)	2,55 (174,3)

Table 6-30 (continued)

Floor	z	Windward	Leeward
	(ft)	kN/m (lb/ft)	kN/m (lb/ft)
26	312	3,91 (267,8)	2,55 (174,3)
27	324	3,95 (270,7)	2,55 (174,3)
28	336	3,99 (273,5)	2,55 (174,3)
29	348	4,03 (276,3)	2,55 (174,3)
30	360	2,04 (139,5)	1,27 (87,17)

6.8.2 Test Results

The results of the eight runs are tabulated in Table 6-31 in terms of the minimum weight and volume of the frame attained in each run. The design history curves obtained in these runs are plotted in Figure 6.24. It is seen from Table 6-31 or Figure 6.24 that the best design weight of 5,250,665.2lb (2,381,701.74kg) is obtained when the frame is sized for minimum weight under initial choice of column orientations. When column orientations are permitted to vary in the optimization process, the best design weight of the frame has been improved to 5,040,628.16lb (2,286,428.94kg). The sectional designations attained for member groups in both design cases are compared in Table 6-32. The optimal layout of the column orientation achieved in the latter case is shown in Figure 6.25. It follows that the optimal layout of column orientations can lead to a reduction as much as 3.6 % in the frame weight.

Table 6-31 The minimum design weights and volumes obtained for 3590-member frame in eight runs.

	Columns with fixed orientations			Columns with varying orientations				
	Test1	Test2	Test3	Test1	Test2	Test3	Test4	Test5
Total Weight	5312947,4	5365007,2	5250665,2	5040628,2	5041610,1	5063400,7	5148126,3	5167920,7
Total Volume	18773666	18957622,6	18553587	17811407	17814876,6	17891875,3	18191259,1	18261203,9

Table 6-32 A comparison of best designs obtained for 3590-member steel frame.

Group Number	Best design with initial column orientations		Best design with varying column orientation	
	Ready Section	Area (cm^2) (in^2)	Ready Section	Area (cm^2) (in^2)
1	W40X277	524,52 (81,3)	W12X53	100,64 (15,6)
2	W33X241	457,42 (70,9)	W36X182	345,81 (53,6)
3	W36X160	303,23 (47)	W27X94	178,71 (27,7)
4	W24X250	474,19 (73,5)	W8X58	110,32 (17,1)
5	W21X166	314,84 (48,8)	W24X131	248,39 (38,5)
6	W14X145	275,48 (42,7)	W14X193	366,45 (56,8)
7	W40X149	282,58 (43,8)	W27X258	488,39 (75,7)
8	W27X178	337,42 (52,3)	W18X234	443,87 (68,8)
9	W33X567	1071 (166)	W30X433	819,4 (127)
10	W30X124	235,48 (36,5)	W21X57	107,74 (16,7)
11	W18X119	226,45 (35,1)	W24X76	144,52 (22,4)
12	W14X99	187,74 (29,1)	W14X90	170,97 (26,5)
13	W40X324	614,84 (95,3)	W30X326	617,42 (95,7)
14	W10X39	74,19 (11,5)	W8X31	58,903 (9,13)
15	W14X90	170,97 (26,5)	W12X58	109,68 (17)
16	W12X96	181,94 (28,2)	W10X60	113,55 (17,6)
17	W21X182	345,81 (53,6)	W14X176	334,19 (51,8)
18	W12X210	398,71 (61,8)	W8X48	90,97 (14,1)
19	W18X175	330,97 (51,3)	W36X182	345,81 (53,6)
20	W40X199	376,77 (58,4)	W14X53	100,64 (15,6)
21	W40X324	614,84 (95,3)	W14X68	129,03 (20)
22	W14X159	301,29 (46,7)	W14X176	334,19 (51,8)
23	W24X131	248,39 (38,5)	W18X158	298,71 (46,3)
24	W12X152	288,39 (44,7)	W33X201	381,29 (59,1)
25	W21X248	469,68 (72,8)	W27X217	411,61 (63,8)
26	W36X439	825,8 (128)	W33X387	729 (113)
27	W16X77	145,81 (22,6)	W10X39	74,19 (11,5)

Table 6-32 (continued)

Group Number	Best design with initial column orientations		Best design with varying column orientation	
	Ready Section	Area (cm^2) (in^2)	Ready Section	Area (cm^2) (in^2)
28	W12X106	201,29 (31,2)	W12X72	136,13 (21,1)
29	W21X101	192,26 (29,8)	W18X97	183,87 (28,5)
30	W12X230	436,77 (67,7)	W40X328	621,93 (96,4)
31	W21X44	83,87 (13)	W10X30	57,032 (8,84)
32	W14X90	170,97 (26,5)	W14X48	90,97 (14,1)
33	W14X99	187,74 (29,1)	W12X45	85,16 (13,2)
34	W14X132	250,32 (38,8)	W12X136	257,42 (39,9)
35	W36X135	256,13 (39,7)	W14X99	187,74 (29,1)
36	W30X99	187,74 (29,1)	W30X191	361,93 (56,1)
37	W18X40	76,13 (11,8)	W14X34	64,52 (10)
38	W18X158	298,71 (46,3)	W14X109	206,45 (32)
39	W18X143	271,61 (42,1)	W30X124	235,48 (36,5)
40	W14X132	250,32 (38,8)	W27X161	305,81 (47,4)
41	W27X146	276,77 (42,9)	W14X159	301,29 (46,7)
42	W36X260	493,55 (76,5)	W14X145	275,48 (42,7)
43	W30X526	993,5 (154)	W27X368	696,8 (108)
44	W8X40	75,48 (11,7)	W27X146	276,77 (42,9)
45	W14X109	206,45 (32)	W16X77	145,81 (22,6)
46	W10X88	167,1 (25,9)	W12X79	149,68 (23,2)
47	W24X229	433,55 (67,2)	W36X245	465,16 (72,1)
48	W8X24	45,677 (7,08)	W14X99	187,74 (29,1)
49	W12X106	201,29 (31,2)	W14X48	90,97 (14,1)
50	W24X94	178,71 (27,7)	W16X40	76,13 (11,8)
51	W21X111	210,97 (32,7)	W14X90	170,97 (26,5)
52	W40X199	376,77 (58,4)	W8X58	110,32 (17,1)
53	W21X83	156,77 (24,3)	W33X141	268,39 (41,6)
54	W14X43	81,29 (12,6)	W12X40	76,13 (11,8)
55	W16X77	145,81 (22,6)	W27X94	178,71 (27,7)

Table 6-32 (continued)

Group Number	Best design with initial column orientations		Best design with varying column orientation	
	Ready Section	Area (cm^2) (in^2)	Ready Section	Area (cm^2) (in^2)
56	W18X130	246,45 (38,2)	W18X119	226,45 (35,1)
57	W10X100	189,68 (29,4)	W27X102	193,55 (30)
58	W10X100	189,68 (29,4)	W30X108	204,52 (31,7)
59	W40X192	364,52 (56,5)	W40X215	408,39 (63,3)
60	W30X357	671 (104)	W24X306	579,35 (89,8)
61	W6X20	37,871 (5,87)	W14X38	72,26 (11,2)
62	W12X72	136,13 (21,1)	W16X67	127,1 (19,7)
63	W10X68	129,03 (20)	W10X88	167,1 (25,9)
64	W14X132	250,32 (38,8)	W14X132	250,32 (38,8)
65	W8X15	28,645 (4,44)	W6X16	30,581 (4,74)
66	W18X46	87,1 (13,5)	W10X39	74,19 (11,5)
67	W24X84	159,35 (24,7)	W10X39	74,19 (11,5)
68	W14X48	90,97 (14,1)	W14X53	100,64 (15,6)
69	W14X99	187,74 (29,1)	W30X108	204,52 (31,7)
70	W16X77	145,81 (22,6)	W10X33	62,645 (9,71)
71	W12X106	201,29 (31,2)	W27X94	178,71 (27,7)
72	W30X173	327,74 (50,8)	W18X130	246,45 (38,2)
73	W36X182	345,81 (53,6)	W27X161	305,81 (47,4)
74	W40X298	565,16 (87,6)	W12X279	528,39 (81,9)
75	W14X61	115,48 (17,9)	W14X26	49,613 (7,69)
76	W10X68	129,03 (20)	W18X50	94,84 (14,7)
77	W12X79	149,68 (23,2)	W10X77	145,81 (22,6)
78	W44X224	424,52 (65,8)	W12X96	181,94 (28,2)
79	W12X58	109,68 (17)	W30X173	327,74 (50,8)
80	W12X40	76,13 (11,8)	W10X33	62,645 (9,71)
81	W18X130	246,45 (38,2)	W30X99	187,74 (29,1)
82	W14X120	227,74 (35,3)	W12X87	165,16 (25,6)
83	W36X135	256,13 (39,7)	W40X167	316,77 (49,1)

Table 6-32 (continued)

Group Number	Best design with initial column orientations		Best design with varying column orientation	
	Ready Section	Area (cm^2) (in^2)	Ready Section	Area (cm^2) (in^2)
84	W40X244	462,58 (71,7)	W40X215	408,39 (63,3)
85	W5X19	35,742 (5,54)	W10X22	41,871 (6,49)
86	W12X87	165,16 (25,6)	W16X40	76,13 (11,8)
87	W21X101	192,26 (29,8)	W12X72	136,13 (21,1)
88	W14X53	100,64 (15,6)	W14X53	100,64 (15,6)
89	W12X40	76,13 (11,8)	W14X53	100,64 (15,6)
90	W44X198	374,19 (58)	W24X94	178,71 (27,7)
91	W18X106	200,64 (31,1)	W27X84	160 (24,8)
92	W24X207	391,61 (60,7)	W12X152	288,39 (44,7)
93	W18X35	66,45 (10,3)	W16X36	68,39 (10,6)
94	W14X109	206,45 (32)	W21X73	138,71 (21,5)
95	W10X100	189,68 (29,4)	W16X67	127,1 (19,7)
96	W36X150	285,16 (44,2)	W14X145	275,48 (42,7)
97	W16X67	127,1 (19,7)	W21X111	210,97 (32,7)
98	W30X191	361,93 (56,1)	W14X74	140,64 (21,8)
99	W21X68	129,03 (20)	W24X62	117,42 (18,2)
100	W30X173	327,74 (50,8)	W24X104	197,42 (30,6)
101	W18X55	104,52 (16,2)	W10X15	28,452 (4,41)
102	W18X130	246,45 (38,2)	W21X101	192,26 (29,8)
103	W10X112	212,26 (32,9)	W36X135	256,13 (39,7)
104	W16X77	145,81 (22,6)	W40X149	282,58 (43,8)
105	W21X44	83,87 (13)	W18X35	66,45 (10,3)
106	W8X10	19,097 (2,96)	W8X10	19,097 (2,96)
107	W18X35	66,45 (10,3)	W18X35	66,45 (10,3)
108	W10X12	22,839 (3,54)	W8X10	19,097 (2,96)
109	W18X35	66,45 (10,3)	W21X50	94,84 (14,7)
110	W12X14	26,839 (4,16)	W33X130	247,1 (38,3)
111	W21X44	83,87 (13)	W24X55	104,52 (16,2)

Table 6-32 (continued)

Group Number	Best design with initial column orientations		Best design with varying column orientation	
	Ready Section	Area (cm^2) (in^2)	Ready Section	Area (cm^2) (in^2)
112	W10X15	28,452 (4,41)	W12X14	26,839 (4,16)
113	W21X83	156,77 (24,3)	W16X45	85,81 (13,3)
114	W24X84	159,35 (24,7)	W14X22	41,871 (6,49)
115	W16X40	76,13 (11,8)	W16X40	76,13 (11,8)
116	W8X15	28,645 (4,44)	W14X22	41,871 (6,49)
117	W16X40	76,13 (11,8)	W18X46	87,1 (13,5)
118	W16X45	85,81 (13,3)	W10X17	32,193 (4,99)
119	W18X50	94,84 (14,7)	W18X50	94,84 (14,7)
120	W6X15	28,581 (4,43)	W12X16	30,387 (4,71)
121	W33X118	223,87 (34,7)	W24X55	104,52 (16,2)
122	W12X26	49,355 (7,65)	W33X118	223,87 (34,7)
123	W24X55	104,52 (16,2)	W33X130	247,1 (38,3)
124	W24X55	104,52 (16,2)	W12X14	26,839 (4,16)
125	W14X233	441,93 (68,5)	W40X298	565,16 (87,6)
126	W14X283	537,42 (83,3)	W14X233	441,93 (68,5)
127	W14X257	487,74 (75,6)	W14X233	441,93 (68,5)
128	W14X159	301,29 (46,7)	W14X176	334,19 (51,8)
129	W14X90	170,97 (26,5)	W14X109	206,45 (32)
Weight	2381701.735kg (5250665.2lb)		2,286,428.94kg(5,040,628.16lb)	

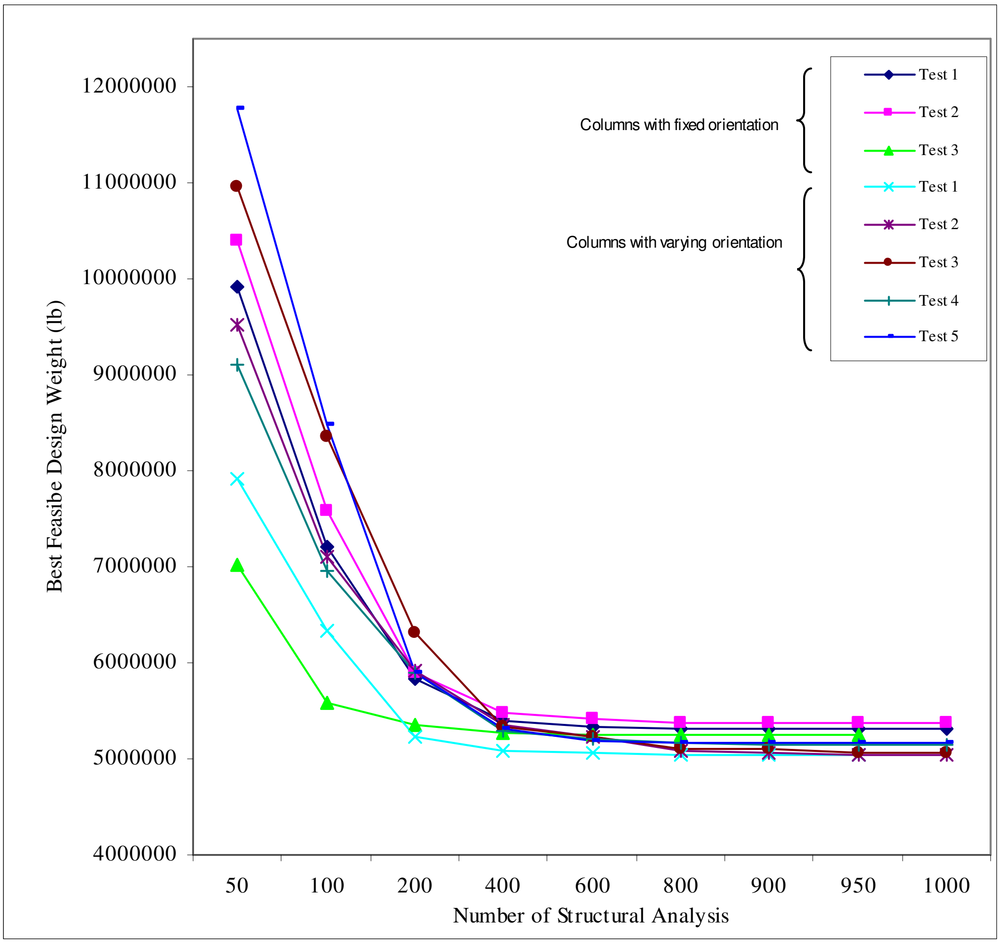


Figure 6.24 Design history graph of 3590 member steel frame

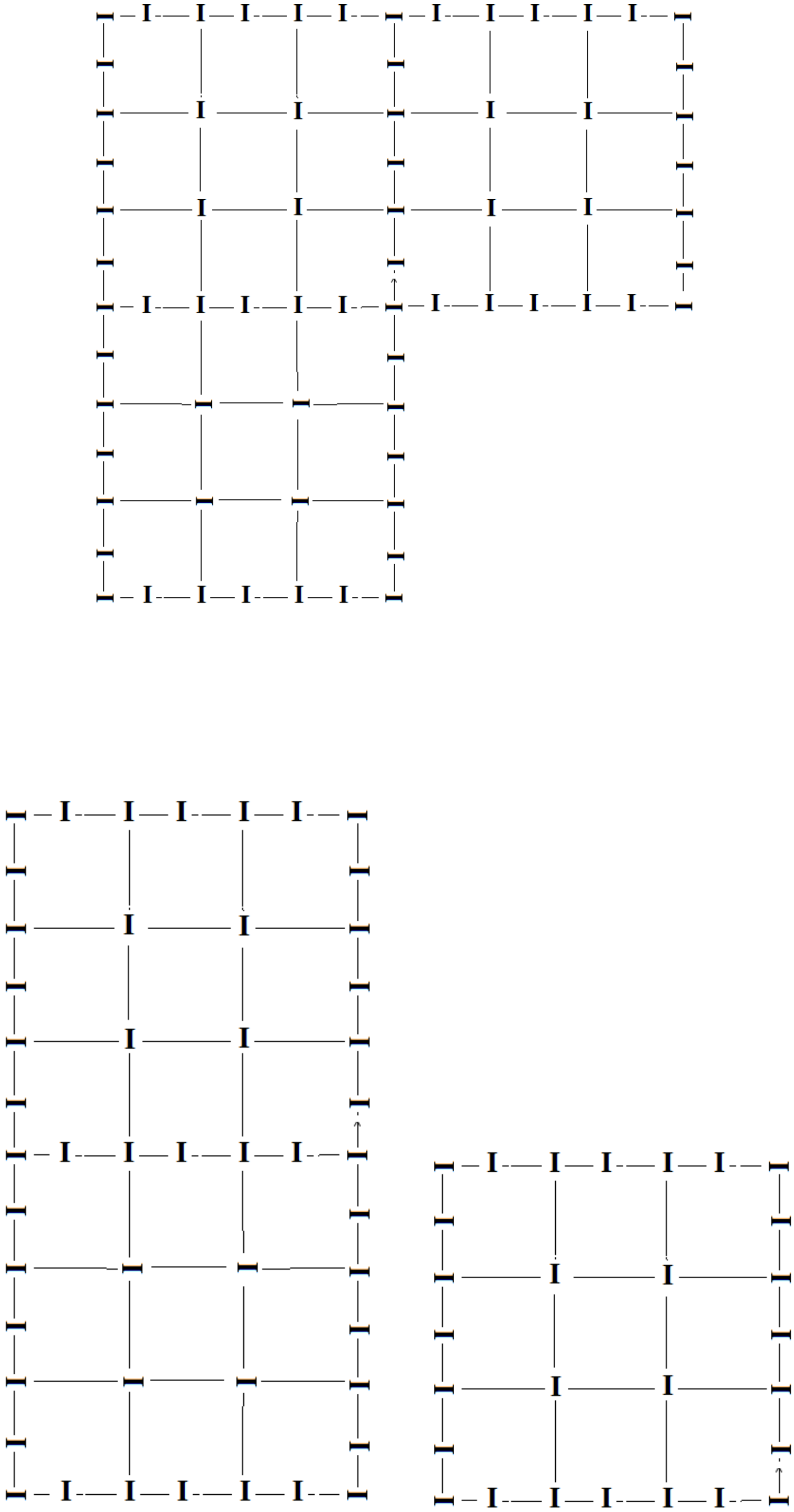


Figure 6.25 Optimal layout column orientation for 3590-member steel frame.

6.9 RESULTS AND EVALUATION

Four different problems are handled in the scope of this study. Each of the structures is subjected to both gravity and lateral loads. For all of the structures, wind load is applied as the lateral loads. The members of the structures are grouped according to the aforementioned criterion. Each of the systems is designed under two main conditions. In the first case only size variables are taken into consideration and optimum weights are obtained according to the initially chosen column orientations. For the second group of runs, orientation variables are also taken as design variables.

For 960 member steel frame example, different wind loads are applied to the same structure. The results revealed that an optimal orientation of the columns results in structural designs which are about 4% to 8.5% lighter than the designs that are based on initial column orientations chosen. 568 member steel frame is analyzed for two different design cases. For the first design case, inclusion of orientation variables results in a reduced structural weight which is 4.1% lighter than the case where members are only sized without a change in their orientations. For the second design case, a reduction of 3.0% in the weight of the structure is achieved by changing the orientations of the members. The results of the two design cases are also differentiate in orientation of columns. As the torsional moments are included, the orientations of the columns are changed in a manner to satisfy the maximum torsional stiffness in the direction of the moment. Figure 6.16. Similarly, for 1230 member steel frame example, the optimal layout of column orientations leads to a reduction as much as 6.0 % in the frames' weights. For the last design example, 3590 member steel frame, a 3.6% reduction in structural weight has been achieved with the proper orientations of the columns. In general, for all of the design examples considered in the study, a weight reduction between 4 % to 8 % has been achieved when orientations of the columns are optimized during the design process.

The column orientations can be predicted according to some design heuristics. If the plan of the structure is square or almost square the system needs to have same rigidity against both directions. This can be satisfied with column orientations. The number of columns whose strong axes are in the same direction with x axis of the structure should be equal to the one's that are in y direction. By this way the building's resistance to the lateral loads will be similar in both directions. In the 960 member steel frame structure, for all of the different wind loads, 4 of the 7 column groups are placed in one direction while the remaining ones are oriented in opposite direction (Figure 6.5, Figure 6.7 and Figure 6.10). There is a similar tendency in the 568 member steel frame structure. For the first three floors, there are totally 7 column groups and the strong axes of 4 of the columns are placed perpendicular to x direction while the 3 of them are oriented in the other direction (Figure 6.14, Figure 6.16). The orientations of the columns in the first three floors are also the same in the upper parts of the structure since all the successive columns along the frame height must have the same orientations. This results with a 2 groups of columns that are orientated in local 3-3 direction and 2 of them that are orientated in local 2-2 direction. In case of a rectangular building, columns are placed with their strong axis perpendicular to short side in order to increase resistance of the building against bending along short side. The columns are placed with their strong axis perpendicular to short side. 1230 member steel frame structure is an example for the tendency of the column orientation in a rectangular building (Figure 6.21). However, as for the non-symmetrical structures such as 3590 member steel frame structure, a generalization is hard to do; the optimization program chooses the most appropriate orientation of each column according to the behavior of the structure. Intuition of the designer might not be enough to place the columns in the correct orientation.

Column orientations of the symmetrical systems that are square in plan view show a similarity in the investigated problems. In both of the directions, outer columns of the frames are oriented on their strong axis on the frame direction

(Figure 6.5, Figure 6.7, Figure 6.10, Figure 6.14). There is an exception about the corner columns. The orientation of inner columns differs in order to satisfy similar stiffness in both of the directions. There is logic behind this orientation. This can be explained basically by the cantilever method. Under the effect of lateral loads, the structure behaves like a cantilever and the outer columns resist to tension and compression forces Figure 6.26, Figure 6.27. This means that most of the axial forces are taken by the outer columns. The moment that results from the lateral loads are shared according to the stiffness of the columns Figure 6.26, Figure 6.27. It is already mentioned that the stress on the frame is the combination of the ratio of axial force to section area and the ratio of moments to section modulus Equation (4.3). It is reasonable to rotate the outer columns on their strong axis. This will help to decrease the total stress on the member by strengthening the section modulus of the member. The profile that is assigned to the member will be used efficiently with correct orientation.

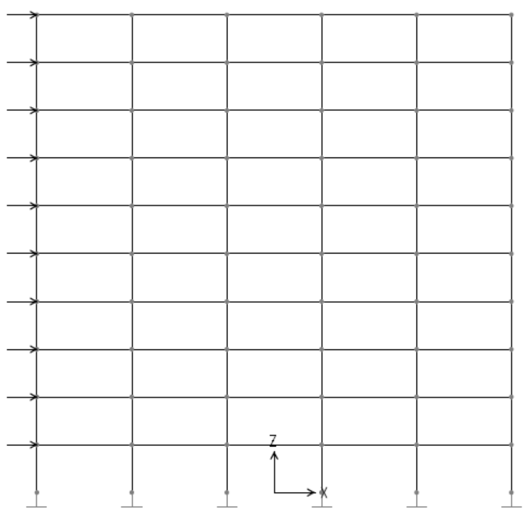
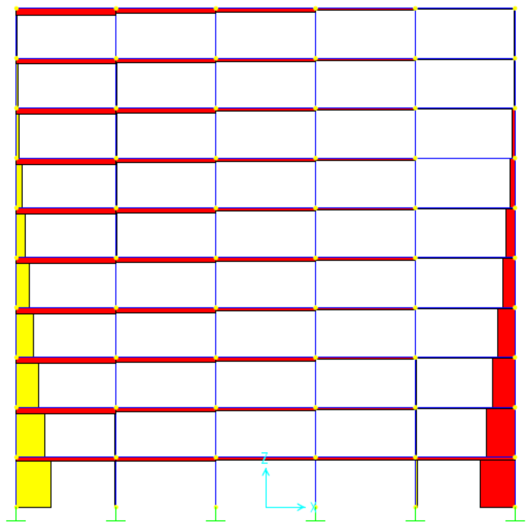
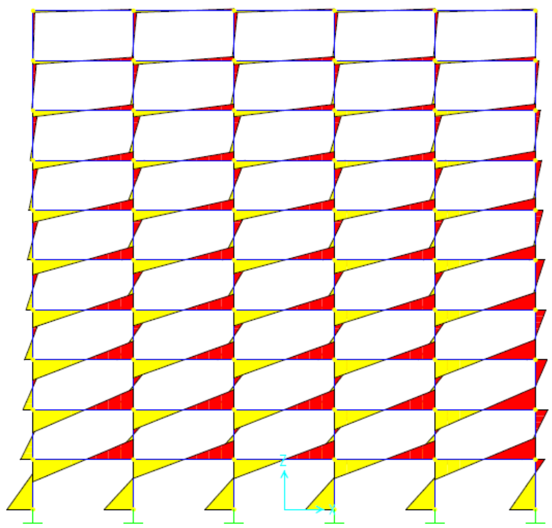


Figure 6.26 Lateral loading of a general steel frame



(a) Axial Force Diagram



(b) Moment Diagram

Figure 6.27 Behavior of a frame under lateral loading

CHAPTER 7

CONCLUSION

7.1 CONCLUSIONS AND SUGGESTIONS

The optimum design process of the steel frames in the literature is only based on sizing of structural members in which orientations of the columns are determined initially and kept constant during optimum design process. In this study, the effect of the appropriate choice of column orientation on minimum weight design of steel frames is investigated. Evolution strategies is used as a tool for sizing optimization of the steel structures. In addition, size optimization is integrated with orientation optimization to further reduce the weight of the structures under certain restrictions and constraints imposed by design specifications. Each of the systems is designed under two main conditions. In the first case only size variables are taken into consideration and optimum weights are obtained according to the initially chosen column orientations. For the second group of runs, orientation variables are also taken as design variables.

The results revealed that an optimal orientation of the columns results in structural designs which are about 4% to 8% lighter than the designs that are based on initial column orientations chosen.

Design heuristics may be in benefit of the designer while defining the initial column orientations of the structures. In case of a structure that's plan view is square or almost square the system needs to have same rigidity against both directions. Equivalent rigidity in both of the directions can be satisfied with column orientations. The number of columns whose strong axes are in the same direction with x axis of the structure should be equal to the one's that are in y direction. The resistance of the structure to lateral loads in both directions will be similar with this orientation. If the plan view of the system is rectangular, the columns are placed with their strong axis perpendicular to short side. By this way the resistance of the building is increased against bending along short side. A generalization is hard to do for non-symmetrical structures. The appropriate column orientation of the structure is assigned by the program according to the behavior of the structure. Intuition of the designer might not be enough to place the columns in the correct orientation.

A similarity is revealed for the investigated problems on column orientations. For the symmetrical systems the outer columns are orientated in the same way. The outer columns of the structure are oriented on their strong axis on the frame direction. However, as for the corner columns a generalization is hard to do. The inner corner columns are oriented in both of the directions to satisfy similar stiffness in both of the directions.

7.2 RECOMMANDATIONS FOR FUTURE STUDIES

The following related subjects shall be investigated in the future studies:

- The effect of column orientation to optimization in industrial structures.
- The effect of column orientation to optimization with additional design provisions like TEC-07.
- The effect of inclusion of geometric constraints to structural optimization.
- The design heuristics of column orientations of non-symmetrical structures.
- Performance of other metaheuristic search techniques with the inclusion of orientation variables to design variables.

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