OPTIMAL WIND BRACING SYSTEMS FOR MULTI-STOREY STEEL BUILDINGS

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ABSTRACT

OPTIMAL WIND BRACING SYSTEMS FOR MULTI-STOREY STEEL BUILDINGS

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The major concern in the design of the multi-storey buildings is the structure to have enough lateral stability to resist wind forces. There are different ways to limit the lateral drift. First method is to use unbraced frame with moment-resisting connections. Second one is to use braced frames with moment-resisting connections. Third one is to use pin-jointed connections instead of moment-resisting one and using bracings. Finally braced frame with both moment-resisting and pin-jointed connections is a solution.

There are lots of bracing models and the designer should choose the appropriate one. This thesis investigates optimal lateral bracing systems in steel structures. The method selects appropriate sections for beams, columns and bracings, from a given steel section set, and obtains a design with least weight. After obtaining the best designs in case of weight, cost analysis of all structures are carried out so that the most economical model is found. For this purpose evolution strategies optimization method is used which is a member of the evolutionary algorithms search techniques.

First optimum design of steel frames is introduced in the thesis. Then evolution strategies technique is explained. This is followed by some information about design loads and bracing systems are given. It is continued by the cost analysis of the models. Finally numerical examples are presented. Optimum designs of three different structures, comprising twelve different bracing models, are carried out. The calculations are carried out by a computer program (OPTSTEEL) which is recently developed to achieve size optimization design of skeletal structures.

Keywords: Optimization, Structural Optimization, Evolutionary Algorithms, Evolution Strategies, Steel Frames, Optimal Bracing Systems.

ÖZ

ÇOK KATLI ÇELİK BİNALARIN OPTİMUM RÜZGAR BAĞLANTI TASARIMLARI

Yıldırım, İlyas

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Çok katlı çelik yapıların tasarımındaki en önemli nokta yapının, rüzgar yüklerine dayanabilmesi için, yeterli yatay kararlılığa sahip olmasıdır. Binalardaki yatay ötelenmeyi belirli sınırlar içinde tutmak için farklı yollar mevcuttur. Birinci yol, bütün kolon kiriş bağlantılarını rijit yapmaktır. İkinci yöntem bu yapılara bir de yatay çapraz elemanları monte etmektir. Üçüncü yol kolon kiriş bağlantılarını mafsallı yapmak ve yatay çapraz elemanları kullanmaktır. Son yol ise hem rijit hem de mafsallı bağlantıları kullanmak ve yatay çapraz elemanları kullanmaktır.

Birçok çapraz modeli mevcuttur ve tasarımcının bunların arasından en uygun olanını seçmelidir. Bu tez, çelik yapıların optimum çapraz tasarımlarını anlatmaktadır. Metod, kirişler, kolonlar ve çapraz elemanları için en uygun kesiti, daha önceden tanımlanmış kesitler listesinden, seçer ve en hafif dizaynı bulur. Daha sonra bütün modeller için maliyet analizleri yapılmaktadır ve böylece en ekonomik model bulunabilmektedir. Bu amaç için, evrimsel algoritmalar üyesi olan, evrimsel stratejiler kullanılır.

İlk başta çelik yapıların optimum tasarımları anlatılmaktadır. Daha sonra evrimsel algoritmalara değinilmiştir. Onu takiben, rüzgar yükleri ve çapraz modelleriyle ilgili bazı bilgiler verilmiştir. Daha sonra yapıların maliyet analizleri anlatılmaktadır. Son olarak da bazı örnek problemler sunulmuştur. Üç farklı yapının, oniki farklı çapraz modeli kullanılarak analizleri yapılmıştır. Bu prosedür, son yıllarda geliştirilen ve yapıların kesit opitmizasyonunu sağlayan, OPTSTEEL ile sürdürülmüştür.

Anahtar Kelimeler: Optimizasyon, Yapı Optimizasyonu, Evrimsel Algoritmalar, Evrimsel Stratejiler, Çelik Yapılar, Optimum Çapraz Tasarımı.

To Kaan

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Chapter 1

INTRODUCTION

The major concern in the design of multi-storey buildings is the lateral stiffness because it is the parameter that controls the lateral drift in the buildings. The lateral stiffness of a building is controlled by different structural systems. These are:

- Using unbraced frame with moment-resisting connections.
- Using braced frame with moment-resisting connections.
- Using braced frame with pin-jointed connections.
- Using braced frame with both moment-resisting and pin-jointed connections at the same time.

Sample bracing for a structure can be seen in Figure 1.1.

Figure 1.1 Bracing

There are lots of possible bracing models. The designer should consider all possible bracing types before deciding. Bracing can be in x-type, v-type, k-type, knee-type, z-type.

Since there are lots of possibilities of bracings and connections, choosing the appropriate one and minimizing the structure cost is the major concern in the design of steel buildings. In Chapter 4 these bracing types are explained. This thesis is mainly focused on the optimal lateral bracing systems in steel structures under wind forces.

The main reason for choosing the wind forces as the main source of lateral force is that the most severe damages in multi-storey steel structures are caused by winds. For example, in the United States between 1986 and 1993, hurricanes and tornadoes caused $41 billion in insured catastrophic losses, compared with $6.18 billion for all other natural hazards combined (Taranath, Wind and Earthquake Resistant Buildings, 2005). In Chapter 3 detailed descriptions of the wind loads are given. Calculation of the wind loads on structures according to ASCE 7-05 is also explained in the chapter.

The optimization process is carried by a method called evolution strategies, which is a member of the evolutionary algorithms search techniques. Evolution strategies were developed in the early 1960s by Rechenberg and Schwefel, who were working at the Technical University of Berlin on an application concerning shape optimization (Eiben & Smith, 1998).

In the latter chapters descriptions of evolutionary algorithms, evolution strategies, wind loads and bracing systems will be given.

1.1 Previous Studies

Another publication related with this topic is Memari & Madhkhan (1999), which deals with the optimization of structures under seismic. Allowable stress design of two-dimensional braced and unbraced steel frames based on AISC specifications subject to gravity and seismic lateral forces is formulated as a structural optimization problem. The objective function is the weight of the structure, and behavior constraints include combined bending and axial stress, shear stress, buckling, slenderness and drift. Cross-sectional areas are used as design variables. Among the specific observations resulting from the examples given in this study is insight with respect to the superiority of the K-braces over the other types in a minimum weight design sense.

One publication is Kameshki & Saka (2001), which uses genetic algorithm for the optimum design of bracing. In this study, the serviceability and stress constraints given in BS 5950 (1990) is used. The algorithm is used to design tall frames with bracings such as X, V and Z bracing. The main difference from our study is that the frames were planar. The technique allows members grouping and selects the required steel sections for beams, columns and bracing members from a set of standard steel sections. The frame used for the optimization procedure is three-bay, 15-storey frame. It is shown that X-bracing system with pinned beam-column connections produces lightest frame.

In Pavlovcic, Krajnc, & Beg (2004) cost function analysis in the structural optimization of steel frames is presented. It is outlined that generally the fabrication and erection costs of the structures are disregarded. The minimum weight does not always mean the most economical solution in the structural design. Therefore, they developed a cost function for the steel frames and this includes all essential fabrication and erection activities. It considers both manufacturing costs as well as material costs. It is formulated in an open manner, offering users to define their own parameters on the basis of a certain production line.

In Hasançebi (2008) the computational performance of adaptive ESs in large-scale structural optimization is mainly investigated to achieve following objectives:

- To present an ESs based solution algorithm for efficient optimum design of large structural systems consisting of continuous, discrete and mixed design variables.
- To integrate new parameters and methodologies into adaptive ESs to improve the computational performance of the algorithm.
- to assess successful self-adaptation models of ESs in continuous and discrete structural optimizations

Two numerical examples are studied in depth to verify the enhanced performance of the algorithm, as well as to scrutinize the role and significance of self-adaptation in ESs for a successfully implemented optimization process. The conclusions of the study are: (i) continuous optimization problems are best handled using $n_\sigma = n_c$, and $n_\sigma = 1$ should be avoided due to its deficient self-adaptation capability, (ii) for discrete optimization problems the learning process works better, if the algorithm is implemented with $n_p = 1$, (iii) the utilization of p_{min} and σ_{min} parameters is crucial for a successfully implemented optimization process, and greatly improves the convergence velocity of the algorithm (iv) a remarkable increase in the efficiency of the algorithm can be gained through the adaptive penalty function implementation.

1.2 Optimization of Steel Frames

In structural design process, there are lots of parameters affecting the design. These are; past experience, tests, research, engineer's capability etc. So for a specific building there is not a single design. Every engineer can design the building in a different way.

The designer makes the initial design, analyzes the building and decides if the building is safe or not. If it is safe enough, he/she can search for a way to design the structure in a more economical way. So the design process is somewhat a trial and error process and every step takes considerable time. The need for the computer software and optimization techniques arises from this aspect.

So when we use a computer with frame optimization software, we have these advantages:

- We design more economical structures.
- We eliminate the designers that have less design experience since any non-experienced engineer can make good designs by using structural optimization software.
- We gain considerable amount of time since we use computers as a computational source.

Therefore design optimization is a very important task. Because of that, it gains attention and more designers make use of optimization in their works.

1.3 Evolutionary Algorithms

Evolutionary algorithms (EAs) are search methods that are inspired from natural selection and survival of the fittest of the nature. EAs are different from more traditional optimization techniques because they search the design space using a population of solutions, not a single point. At every iteration of an EA, the poor solutions are eliminated. The solutions having high fitness values are recombined with other solutions that have high fitness by interchanging their parts. These solutions are also mutated by making some changes to their elements. To generate new solutions recombination and mutation are used, and it is hoped that these new solutions move towards regions where the good solutions lie. A brief flowchart for an EA is as follows:

- Initialize the population
- Evaluate initial population
- Apply genetic operators to generate new solutions
- Evaluate solutions in the population
- Perform competitive selection
- Repeat, above three steps, until some convergence criteria is satisfied

This process is iterated until a specified criterion is reached or until a solution that meets the sufficient quality is found.

There are different types of EAs. The major ones are:

- Genetic Algorithms(GAs)
- Evolutionary Programming(EP)
- Evolution Strategies(ESs)

GAs were developed by Holland (University of Michigan) in the early 1960s. In GAs, the solution of a problem is encoded in the form of strings of numbers (traditionally binary). Both mutation and recombination are used in GAs and the evaluation is made by using a fitness function.

EP was developed by Fogel first. Intelligent behavior was viewed as the ability to predict the next state of the machine environment. When an input symbol is presented to the machine, the machine generates an output symbol, which is compared to the next input symbol. The current output symbol represents a prediction of the next input symbol. The quality of the prediction is measured by using a payoff function (Dumitrescu, Lazzerini, Jain, & Dumitrescu, 2000).

ESs were first developed by Rechenberg (1965) and Schwefel (1981). ESs use natural representation with a vector of real values and main operators are mutation, recombination and selection. The operators are applied in a loop. An iteration of the loop is called a generation. The sequence of generations is continued until a termination criterion is met. Since ESs are used in this study, the brief introduction about ESs will be given in the next section.

1.4 Evolution Strategies

ESs are a branch of direct search and optimization methods that are inspired from the nature, belonging to the class of EAs. ESs use natural problem-dependent representations, and primarily mutation and selection as search operators. The operators are applied in a loop. One iteration of the loop is called a generation. The loops are continued until a termination criterion is met.

It was created in the early 1960s by Rechenberg Schwefel at the Technical University of Berlin, Germany.

At first, ESs were used for solving optimization problems related with fluid dynamics, but later, they were used for solving other optimization problems with discrete variables. As far as the search space is real valued, mutation is normally applied by adding a normally distributed random value to each vector component.

The simplest form of the EAs is the (1+1)-ES. In this form the individual creates an offspring and if the offspring's fitness is better or equal than the parent, it becomes the parent of the next generation. Otherwise the offspring is eliminated. The first multi-membered ES is the $(\mu + 1)$-ES. In this form μ parents combine to generate a single offspring and this offsprings replaces the worst parent.

These types are replaced by (μ, λ)-ES and $(\mu + \lambda)$-ES. Here, μ stands for the parents and λ stands for the offsprings. From μ-parents, λ-offsprings are created and these offsprings compete with themselves in (μ, λ)-ES, i.e. the parents are eliminated. In $(\mu + \lambda)$-ES however, the parents are not eliminated so both offsprings and the parents are in a competition. In Bäck (1996), detailed explanations of these selection types and their drawbacks are given.

1.5 Aim and Scope of the Study

The main aim of this thesis is to investigate the effect of bracing system on optimum design of steel structures. Different structures with different bracing systems are sited for minimum weight and cost and the results are compared.

The thesis is organized as follows:

In Chapter 2 design optimization of steel frames using evolution strategies is described. First detailed overview of the optimization subject is given. The elements of the optimizations are mentioned and their details are also given in this chapter. After that the design optimization of steel frames is formulated. Optimization of a steel frame according to ASD-AISC is explained here. The constraints such as stress, displacement, and geometry are formulated and values of them are given. After defining the problem formulation the evolutionary algorithms are introduced. The background information of them and their main elements can be found here. Evolution strategies, being a technique of evolutionary algorithms, are explained then. The background information, different extensions and elements of ESs are given.

Chapter 3 focuses on the design loads on structures. There are four types of loads on our structures. These are dead loads, live loads, snow loads and wind loads. First a brief introduction about the wind loads is given and then the calculation of wind loads on structures according to the ASCE 7-05 is outlined. Then calculation of the other three load types is given then.

Chapter 4 discusses the bracing of steel frames. The advantages of bracing on structures are emphasized and different types of bracing systems are given. The illustrations of the bracing systems that are used in our structures are also available in this chapter.

Chapter 5 is the cost analysis of steel frames. The cost of our structures is mainly assumed to be composed of four elements. These are, cost of elements, cost of joints, cost of transportation and cost of erection. In this chapter the calculation of the cost of a structure is outlined.

In Chapter 6, numerical examples are given. Details of the test problems for the optimum bracing type are explained. There are three different structures. They have 10, 20 and 30 floors respectively. Different bracing types are applied to these structures and optimum designs of them are carried out.

Chapter 7 gives interpretation of the results obtained. The comparison of the bracing types and their behaviors are described here.

Chapter 8 makes a conclusion of the thesis by outlining the results and discussions.

Chapter 2

OPTIMUM DESIGN OF STEEL FRAMES USING EVOLUTION STRATEGIES

2.1 Optimization

Optimization can be described as the process of having the best solution of a given objective(s) while satisfying certain restrictions. It can also be defined as finding the minima or maxima of a given objective function under some constraints.

Being a very widely used phenomenon in today's world, optimization is used to improve business processes in practically all industries such as operations research, artificial intelligence, computer science, and structural design.

There are mainly four elements of optimization. These are:

- Design variables
- Constraints
- Objective function
- Design space

In order to improve a property of a structure, one has to change some parameters of it. These parameters are called design variables. Design variables can be a cross-sectional dimension or simply be a cross-section type from a given set of steel sections. Design variables can take either discrete or continuous values. For instance in case of having a set of steel sections that we can purchase from, we need to use discrete design variables. If we want the variable to assume any value between some boundaries, then we use continuous design variables. Some design variables can also be set to some specific values that are not to be changed during the design process. These parameters are called preassigned parameters.

Constraints are some special conditions that the design must satisfy in order to be accepted as a feasible design. This can be the cross-sectional dimensions of a column to be in a range or a stress in a member not to exceed a given value. There are two different constraint types. These are geometric constraints and behavior constraints. The need for a geometric constraint may arise from fabrication limitations or aesthetics. A typical example for a behavior constraint is the strength of a member.

Objective functions are the functions that measure the quality of the design. Different solutions are compared according to their objective functions. For structural optimization, minimization of cost, weight, displacement, stress or load can be used as objective function. If the problem has a single objective function, it said to be a single-criterion optimization problem. If it has multiple objective functions, it is a multi-criteria optimization problem.

There are mainly three types of structural optimization. These are size optimization, shape (geometry) optimization and topology optimization.

Size optimization deals with changing the dimensions of a given element. Typical example for a size optimization can be seen in Figure 2.1.

Shape optimization deals with changing the shape of the element, such that optimum locations of nodes in a finite element model of the structure are determined.

In topology optimization, the best distribution of material for a given structure is found. An optimal structure is generated by carving out material from a given space, for a given amount of material. Figure 2.2 is an example for a topology optimization.

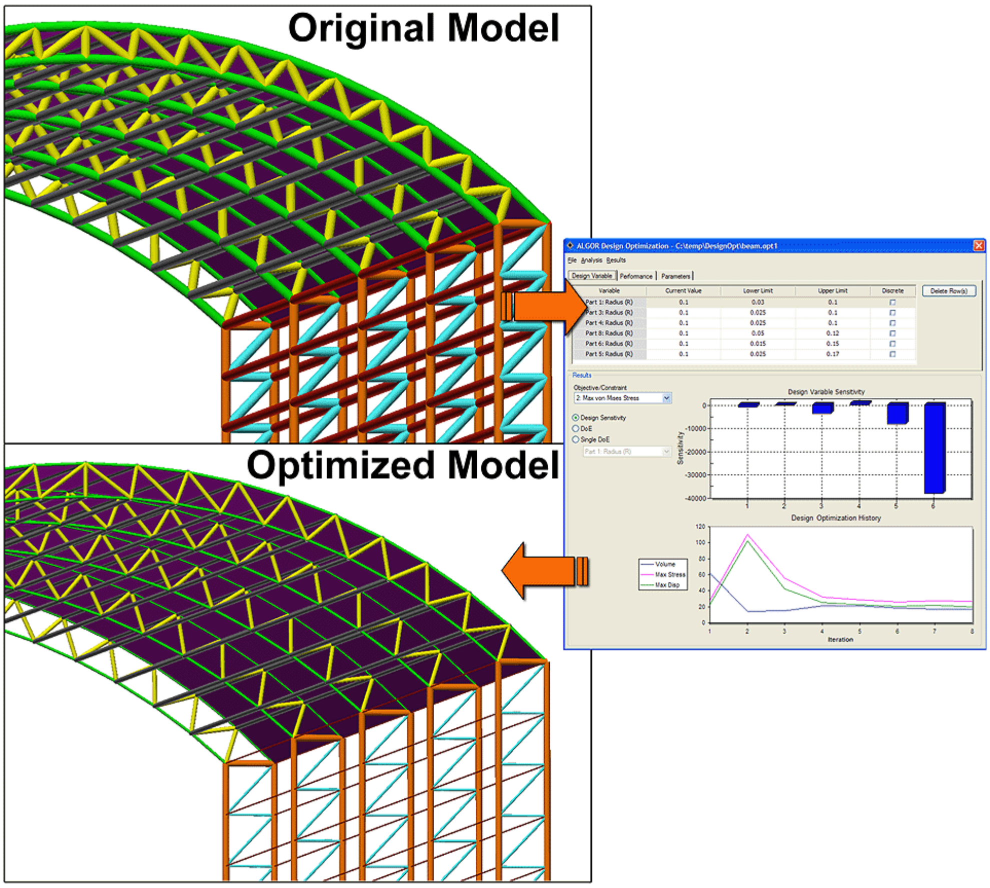


Figure 2.1 Size Optimization (Algor, 2009)

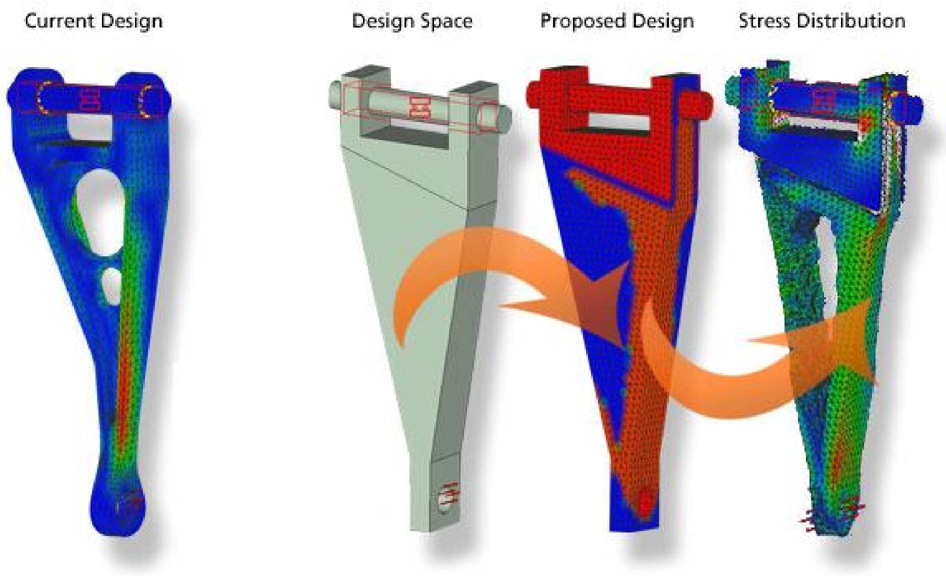


Figure 2.2 Topology Optimization Example (Topologica Solutions, 2009)

2.2 Design Optimization of Steel Frames

For a steel frame structure consisting of N_m members that are collected in N_d design groups (variables), the optimum design problem according to AISC(1989) code yields the following discrete programming problem, if the design groups are selected from steel sections in a given profile list.

Find a vector of integer values $\mathbf{I}$ (Equation (2-1)) representing the sequence numbers of steel sections assigned to N_d member groups

$$\mathbf{I}^{\mathrm{T}} = \left[I_1, I_2, \dots, I_{N_d}\right] \tag{2-1}$$

to minimize the weight (W) of the frame:

$$W = \sum_{i=1}^{N_d} \rho_i A_i \sum_{j=1}^{N_t} L_j \qquad \textbf{(2-2)}$$

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

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

$$\text{if}\ \frac{f_a}{F_a} > 0.15; \left[\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{C_{my} f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_{by}}\right] - 1.0 \le 0 \qquad \textbf{(2-3)}$$

$$\text{and}\ \left[\frac{f_a}{0.6F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}\right] - 1.0 \le 0 \qquad \textbf{(2-4)}$$

$$\text{if}\ \frac{f_a}{F_a} \le 0.15; \quad \left[\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}\right] - 1.0 \le 0 \qquad \textbf{(2-5)}$$

If the flexural member is under tension, then the following formula is used instead:

$$\left[\frac{f_a}{0.6F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}}\right] - 1.0 \le 0 \qquad \textbf{(2-6)}$$

In Equations (2-3) to (2-6):

- F_a stands for the allowable axial stress under axial compression force alone, and is calculated depending on elastic or inelastic bucking failure mode of the member using formulas given in Chapter E of AISC (1989).
- $f_a = (P/A)$ represents the computed axial stress, where A is the cross-sectional area of the member.
- f_{bx} and f_{by} represent the computed flexural stresses due to bending of the member about its major (x) and minor (y) principal exes.
- F_{ex}' and F_{ey}' denote the Euler stresses about principal axes of the member that are divided by a factory of safety of 23/12. The formulation can be found in Chapter H of AISC (1989).
- F_y is the material yield stress
- The allowable bending compressive stresses about major and minor axes are designated by F_{bx} and F_{by} which are computed using the formulas given in Chapter F of AISC (1989).
- C_{mx} and C_{my} are the reduction factors, introduced to counterbalance overestimation of the effect of secondary moments by the amplification factors $(1 - f_a/F_e')$. For braced frame members without transverse loading between their ends, they are calculated from $C_m = 0.6 - 0.4(M_1/M_2)$, where M_1/M_2 is the ratio of smaller end moment to the larger end moment. For braced frame members having transverse loading between their ends, they are determined from the formula $C_m = 1 + \psi\,(f_a/F_e')$ based on a rational approximate analysis outlined in Chapter H of AISC (1989), where ψ is a parameter that considers maximum deflection and maximum moment in the member.

For computation of allowable compression and Euler stresses, the effective length factors K are required. For beam and bracing members, K is taken equal to unity. For column members, alignment charts are furnished in ASD-AISC (1989) for calculation of K values for both braced and unbraced cases. In this study, however, the following approximate effective length formulas are used, which are accurate to within about -1.0 and +2.0 % of exact results:

For unbraced members:

$$K = \sqrt{\frac{1.6G_A G_B + 4(G_A + G_B) + 7.5}{G_A + G_B + 7.5}} \tag{2-7}$$

For braced members:

$$K = \frac{3G_A G_B + 1.4(G_A + G_B) + 0.64}{3G_A G_B + 2.0(G_A + G_B) + 1.28} \tag{2-8}$$

Where G_A and G_B refer to stiffness ratio or relative stiffness of a column at its two ends.

It is also required that computed shear stresses (f_v) in members are smaller than allowable shear stresses (F_v), as formulated in Equation (2-9).

$$f_v \leq F_v = 0.40 C_v F_v \tag{2-9}$$

In Equation (2-9), C_v is referred to as web shear coefficient. It is taken equal to $C_v = 1.0$ for rolled W-shaped members with $h/t_w \leq 2.24E/F_y$, where 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 given in Chapter G of ANSI/AISC (2005).

Apart from stress constraints, slenderness limitations are also imposed on all members such that maximum slenderness ratio ($\lambda = KL/r$ is) limited to 300 for members under tension, and to 200 for members under compression loads.

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.

Finally, we consider geometric constraints between beams and columns framing into each other at a common joint for practicality of an optimum solution generated. For the two beams B1 and B2 and the column shown in Figure 2.3, one can write the following geometric constraints:

$$\frac{b_{f\,b}}{b_{f\,c}} - 1.0 \leq 0 \tag{2-10}$$

$$\frac{b_{f\,b}'}{(d_c - 2t_f)} - 1.0 \leq 0 \tag{2-11}$$

where $b_{f\,b}$ $b_{f\,b}'$ and $b_{f\,c}$ 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 (2-10) simply ensures that the flange width of the beam B1 remains smaller than that of the column. On the other hand, Equation (2-11) enables that flange width of the beam B2 remains smaller than clear distance between the flanges of the column$(d_c - 2t_f)$.

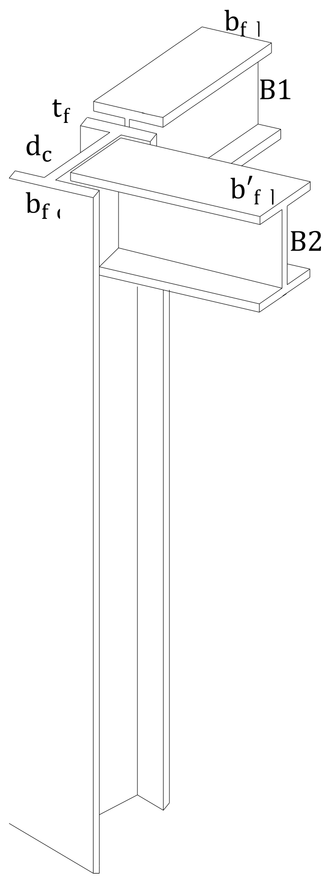


Figure 2.3 Sample Figure for Geometric Constraints

2.3 Evolutionary Algorithms

Evolutionary algorithms is an interdisciplinary research field with a relationship to biology, artificial intelligence, numerical optimization, and decision support in almost any engineering discipline (Bäck, 1996).

There are lots of possible techniques of evolutionary algorithms. The common underlying idea behind all is the same: given a population of individuals, the environmental pressure causes natural selection (survival of the fittest), which causes a rise in the fitness of the population. Given a quality function to be maximized, we can randomly create a set of candidate solutions, i.e., elements of the function's domain, and apply the quality function as an abstract fitness measure – the higher the better. Based on this fitness, some of the better candidates are chosen to seed the next generation by applying recombination and/or mutation to them. Recombination is an operator applied to two or more selected candidates (the so-called parents) and results one or more new candidates (the children). Mutation is applied to one candidate and results in one new candidate. Executing recombination and mutation leads to a set of new candidates (the offspring) that compete – based on their fitness (and possibly age) – with the old ones for a place in the next generation. This process can be iterated until a candidate with sufficient quality (a solution) is found or previously set computational limit is reached (Eiben & Smith, 1998) .

In evolutionary algorithms there are two fundamental forces:

- Variant Operators: Recombination and Mutation.
- Selection: It is the force that pushes quality.

In evolutionary algorithms, although the weak individuals have a chance to be selected as a parent, fitter individuals have a higher chance to be selected.

Evolutionary Algorithms have various elements such as:

- Representation
- Evaluation Function
- Population
- Parent Selection Mechanism
- Variation operators; recombination and mutation
- Survivor Selection Mechanism

This process is used by different techniques, i.e. evolution strategies, genetic algorithms and evolutionary programming. In the next section evolution strategies will be explained.

2.4 Evolution Strategies

The fundamentals of ESs were originally laid in the pioneering studies of Rechenberg (1965, 1973) and Schwefel (1965) at the Technical University of Berlin. They developed the first (simplest) variant of ESs, which implements on the basis of two designs; a parent and an offspring individual. Today, the modern variants of ESs are accepted as $(\mu + \lambda) - ES$ and $(\mu , \lambda) - ES$, which were again developed by Schwefel (1977, 1981). Both variants employ design populations consisting of μ parent and λ offspring individuals, and are intended to carry out a self-adaptive search in continuous design spaces. The extensions of these variants to solve discrete optimization problems were put forward in the following three studies in the literature: Cai and Thierauf (1993), Bäck and Schütz (1995), and Rudolph (1994). Amongst them, the one proposed by Cai and Thierauf (1993) refers to a non-adaptive reformulation of the technique and has probably found the most applications in discrete structural optimization. The approach proposed by Bäck and Schütz (1995) corresponds to an adaptive reformulation of the technique, which incorporates a self-adaptive strategy parameter called mutation probability. Another adaptive reformulation of ESs is presented by Rudolph (1994) for general non-linear mathematical optimization problems.

2.4.1 Optimization Routine

As in all EA techniques, the underlying idea of ESs rests on simulation of natural evolution in an effort to evolve a population of individuals (designs) towards the optimum. In this framework, both continuous and discrete adaptive ESs make use of a common optimization routine featuring a generation-based iteration of the technique. This routine is outlined in the flowchart shown in

Figure 2.4, where $P(t)$ and $P'(t)$ denote the parent and offspring populations at a generation (iteration) t, respectively.

Concerning this flowchart, the first two steps consist of setting the generation counter t to 0 and creating an initial population $P(0)$. The initial population consists of μ parent individuals, which are customarily created through a random initialization. Hence, it is highly 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 the 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 individuals, which also undergo an evaluation process (step 5 in Figure 2.4) to attain a fitness scores. Next, in step 6, the survivors of parent and offspring populations are determined via the selection operator, which identifies the only difference between $(\mu + \lambda)$ and (μ, λ) variants of ESs. In the $(\mu + \lambda) - ES$, the operator is implemented by choosing deterministically the best μ individuals from a sum of $(\mu + \lambda)$ parent and offspring individuals. On the other hand, the $(\mu, \lambda) - ES$ excludes the parent population from the selection mechanism – instead the best μ individuals are chosen only from the λ offspring individuals. This completes one generation in the optimization procedure, accompanied by an increase of the 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 steps 4 and 8 is iterated in the same way for each new value of the generation counter until a termination criterion is satisfied.

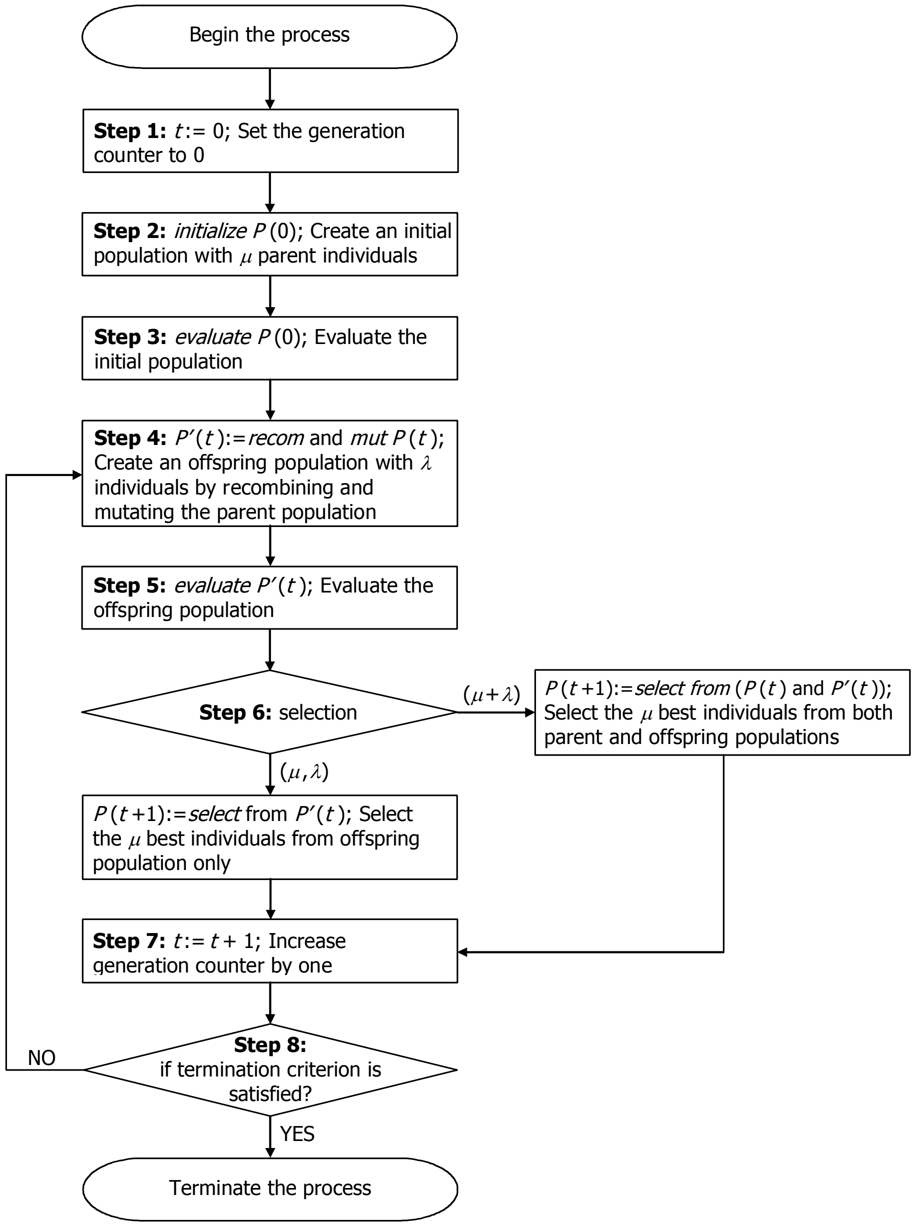


Figure 2.4 Optimization Routine

The optimization routine discussed above forms the basic framework of the solution algorithm developed in the present work.

2.4.2 ESs for Continuous Variables

In this section (μ, λ)-ES formulation for problems having continuous design variables will be given.

2.4.2.1 Representation of an Individual and Initial Population

As the most general form of individuals in ESs, the Equation (2-12) can be used:

$$\vec{a} = (\vec{x}, \vec{\sigma}, \vec{\alpha}) \tag{2-12}$$

Where $\vec{a}$ represents the individual (a possible solution) in ESs. It consists of three components, $\vec{x}$, $\vec{\sigma}$ and $\vec{\alpha}$. $\vec{x}$ represents the design variables vector and it is used to calculate the objective function. $\vec{\sigma}$ and $\vec{\alpha}$ represents the strategy parameter set of $\vec{x}$.

Search points in Evolution Strategies are n-dimensional object parameter vectors $\vec{x} \in \mathbb{R}$ The objective function Φ is in principle identical to the fitness function: $\mathbb{R}^n \to \mathbb{R}$, i.e. given an individual $\vec{a} \in I$, we have;

$$\Phi(\vec{a}) = f(\vec{x}) \tag{2-13}$$

Where $\vec{x}$ is the object variable component of $\vec{a} = (\vec{x}, \vec{\sigma}, \vec{\alpha}) \in I = \mathbb{R} \times A_s$, where;

$$A_s = \mathbb{R}_+^{n_\sigma} \times [-\pi, \pi]^{n_\alpha} \tag{2-14}$$

$$n_\sigma \epsilon \{1, \dots, n\} \tag{2-15}$$

$$n_\alpha \epsilon \{0, (2n - n_\sigma)(n_\sigma - 1)/2\} \tag{2-16}$$

Besides representing the object variable vector $\vec{x}$, each individual may additionally include one up to n different standard deviations σ_i as well as up to $n \cdot (n - 1)/2$ (namely when $n_\sigma = n$) rotation angles $\alpha_{ij} \in [-\pi, \pi]$ $(i \in \{1, \dots, n - 1\}, j \in \{i + 1, \dots, n\})$, such that the maximum number of strategy parameters amounts to $w = n \cdot (n + 1)/2$. For the case $1 < n_\sigma < n$, the standard deviations $\sigma_1, \dots, \sigma_{n_\sigma - 1}$ are coupled with object variables $x_1, \dots, x_{n_\sigma - 1}$ and σ_{n_σ} is used for the remaining variables $x_{n_\sigma}, \dots, x_n$. Here n is the size of the vector of object variables.

The number n_α of rotation angles depends directly on n_σ, the number of standard deviations, and n, but it can also explicitly be set to zero, indicating that this strategy parameter part of individuals is not used (Bäck, 1996).

Initial population consisting of μ parent individuals is created by means of a random initialization. Therefore, for each individual the variables are selected with a uniform distribution between their specified lower and upper bounds. No requirement regarding the feasibility of the individuals is enforced, and thus the initial population is permitted to retain infeasible individuals in addition to feasible ones.

2.4.2.2 Constraint Handling and Evaluation of Population

Not only the objective function, but also the problem constraints have to be taken into account during the evaluation process of parent and offspring individuals. A variety of different approaches and/or specialized operators have been proposed in the literature to handle constraints. The constraints are dealt with using an external penalty function approach in this study. Objective function values of the feasible solutions that satisfy all the problem constraints are directly calculated first. Then the 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:

$$\Phi_c = \Phi[1 + \mathrm{Penalty}(\vec{a})] = \Phi\left[1 + r\left(\sum_{j=1}^{n_j} g_j\right)\right] \tag{2-17}$$

In Equation (2-17), Φ_c is the constrained objective function value. $g_j, j = 1, .., n_d$ represents the whole set of normalized constraints, and r is the penalty coefficient used to tune the intensity of penalization as a whole. Although r can be assigned to an appropriate static value, such as $r = 1$, an adaptive penalty function implementation is favored by letting this parameter adjust its value automatically during the search for the most efficient optimization process. The second method (letting r to adjust itself during search) can be made by:

$$r(t) = \begin{cases} \left(\frac{1}{f}\right) \cdot r(t-1)\ i\ {}_j b(t-1) i\ {:} f\ e\ a\ s\ i \\ \\ f \cdot r(t-1)\ \ i\ {}_j b(t-1) i\ {:} i\ n\ f\ e\ a\ s \end{cases} \tag{2-18}$$

Where $r(t)$ and $r(t-1)$ 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 r. Experiments with various test problems indicate that the optimal value of f equals to 1.1.

The rationale behind Equation (2-18) 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 feasible, 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. Another feature of the adaptive penalty function is that it avoids entrapment of the search at local optimum, which is the case, often observed when a static penalty function is utilized.

2.4.2.3 Recombination

Recombination is applied to create an offspring population, such that μ parent individuals undergo an exchange of design characteristics to produce λ offspring individuals. Several recombination operators exist and, in principle, recombination of different components of an individual can be implemented using different operators. Assuming that $\vec{v}$ represents an arbitrary component of an individual, a formulation of these operators is given in Equation (2-19) as applied to produce $\vec{v}'$

$$v_i' = \begin{cases} v_{a,i}, & \text{no recombination} \\ v_{a,i} \text{ or } v_{b,i}, & \text{discrete} \\ v_{a,i} \text{ or } v_{b_j,i}, & \text{global discrete} \\ v_{a,i} + \dfrac{v_{b,i} - v_{a,i}}{2}, & \text{intermediate} \\ v_{a,i} + \dfrac{v_{b_j,i} - v_{a,i}}{2}, & \text{global intermediate} \end{cases} \tag{2-19}$$

Where v_a and v_b represents the v component of any two parent individuals that are chosen from parent population at random:

- Type (1) corresponds to the no recombination case; v_i' is simply formed by duplicating v_a.

- Type (2) is named as discrete recombination and each element of $\vec{v}'$ is selected from one of the two parents $(v_a\ a\ n\ d_a)$ under equal probability.
- Type (3) is called as 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 $\vec{v}'$, and then v_i' is chosen from one of these parents (v_a, v_{b_j}) under equal probability.
- Intermediate forms of type (2) and (3) are given in types (4) and (5), respectively, which are identical to the former except that arithmetic means of the elements are calculated.

In order to explain the different types of recombination, the example shown in Bäck(1996) can be used:

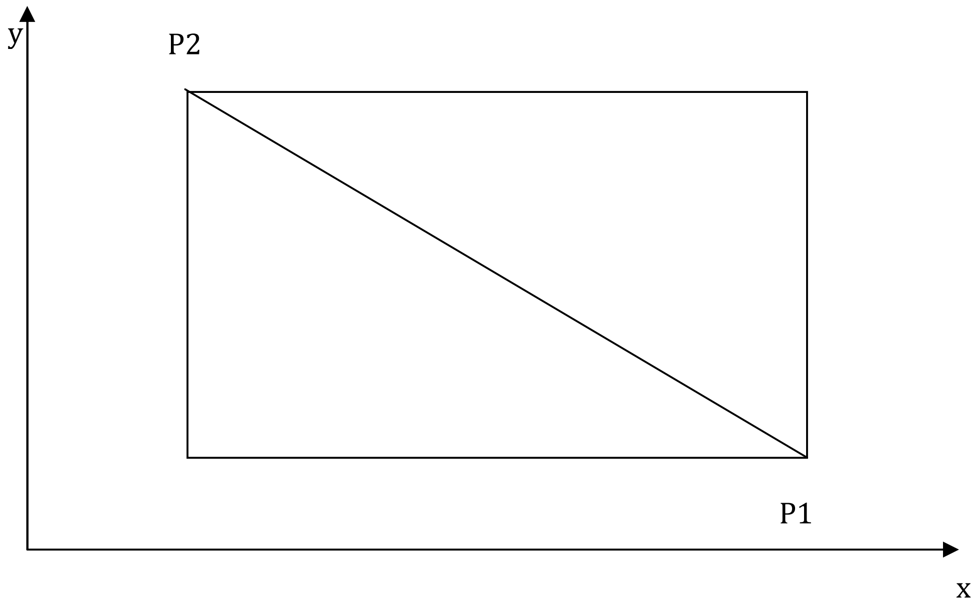


Figure 2.5 Schema of Recombination Types

In Figure 2.5 P1 and P2 represents the object variable points represented by two parent individuals. Only the corners of the rectangle defined by P1 and P2 can be reached by means of discrete recombination (Type (2) and Type (3)). Intermediate recombination (Type (4) and Type (5)) yields the center of the rectangles' diagonal lines.

2.4.2.4 Mutation

The mutation operator in ESs is based on a normal (Gaussian) distribution requiring two parameters: the mean ξ and the standard deviation σ.

In practice, the mean ξ is always set to zero, and the vector $\vec{x}$ is mutated by replacing x_i values by:

$$x_i' = x_i + N(0, \sigma) \tag{2-20}$$

where $N(0, \sigma)$ denotes a random number drawn from a Gaussian distribution with zero mean and standard deviation σ. By using a Gaussian distribution here, it is ensured that small mutations are more likely than larger ones.

What is important here is that the mutation step sizes are not set by the user; rather the σ is coevolving with the solutions. In order to achieve this behavior it is essential to modify the value of σ first, and then mutate the x_i values with the new σ value. The rationale behind this is that a new individual $(\vec{x}', \sigma')$ is effectively evaluated twice. Primarily, it is evaluated directly for its viability during survivor selection based on $f(\vec{x}')$. Second, it is evaluated for its ability to create good offspring. This happens indirectly: a given step size evaluates favorably if the offspring generated by using it prove viable (in the first sense). Thus, an individual $(\vec{x}', \sigma')$ represents both a good $\vec{x}'$ that survived selection and a good σ' that proved successful in generating this good $\vec{x}'$ from $\vec{x}$ (Eiben & Smith, 1998).

Since a general formulation of mutation is given in Equation (2-20), now the three special cases of mutation can be given;

- Uncorrelated Mutation with One Step Size
- Uncorrelated Mutation with n Step Sizes
- Correlated Mutation

2.4.2.4.1 Uncorrelated Mutation with One Step Size

In this case of mutation, the same distribution pattern is used to mutate the values of $\vec{x}$. That means we have only one single value of the strategy parameter σ for each individual. This type of mutation can be expressed as:

$$\sigma' = \sigma \cdot \exp\,(\tau \cdot N(0,1)) \tag{2-21}$$

$$x_i{}' = x_i + \sigma' \cdot N_i(0,1) \tag{2-22}$$

Where $N\ (0,1)$ denotes the standard normal distribution, $N_i(0,1)$ denotes the single standard normal distribution for each variable i. The τ denotes the proportionality constant. It can be explained as the learning rate. It is usually inversely proportional with the square root of the problem size:

$$\tau \propto 1/\sqrt{n} \tag{2-23}$$

The σ values are required not to get close to zero, so a boundary condition is set for the σ, and if it gets below that level, σ is set to that boundary value.

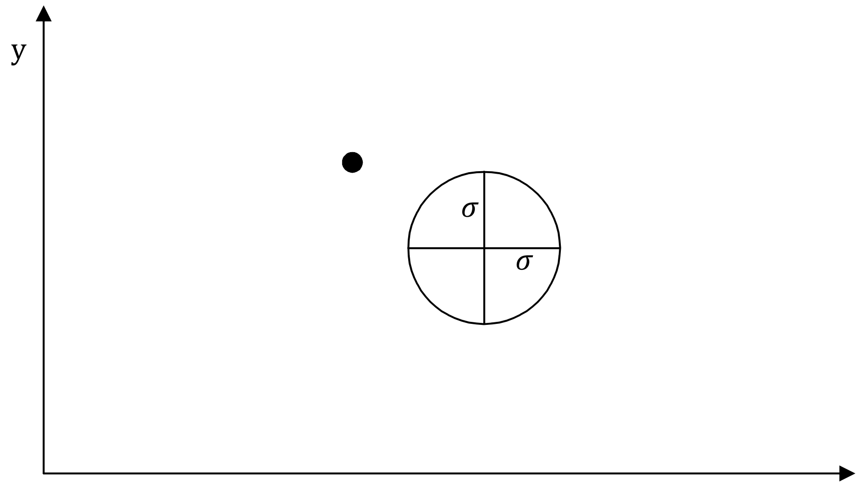


Figure 2.6 Uncorrelated Mutation With One Step Size

Let's assume that we have a population with n=2 and this is an uncorrelated mutation with one step size; $n_\sigma = 1$, $n_\alpha = 0$. In Figure 2.6 the black dot represents the local maximum and the circle indicates the points where the offsprings can be placed with a given probability. The probability of moving along both the x-axis and y-axis are the same.

2.4.2.4.2 Uncorrelated Mutation with n Step Sizes

In this case of mutation, the different distribution patterns are used to mutate the values of $\vec{x}$. The reason for this is that the fitness directions can have different slopes for different directions. That means we have different values of the strategy parameter σ for each variable of an individual. This type of mutation can be expressed as:

$$\sigma_i{}' = \sigma_i \cdot \exp\left[\tau' \cdot N(0,1) + \tau \cdot N_i(0,1)\right] \tag{2-24}$$

$$x_i{}' = x_i + \sigma_i{}' \cdot N_i(0,1) \tag{2-25}$$

where N (0,1)denotes the standard normal distribution, $N_i(0,1)$ denotes the single standard normal distribution for each variable i. The τ and τ' denotes the proportionality constants. They can be described as the learning rate:

$$\tau \propto 1/\sqrt{2\sqrt{n}} \tag{2-26}$$

$$\tau' \propto 1/\sqrt{2n} \tag{2-27}$$

Also, in this case, the σ values are required not to get close to zero, so a boundary condition is set for them, and if they get below that level, they are set to that boundary value.

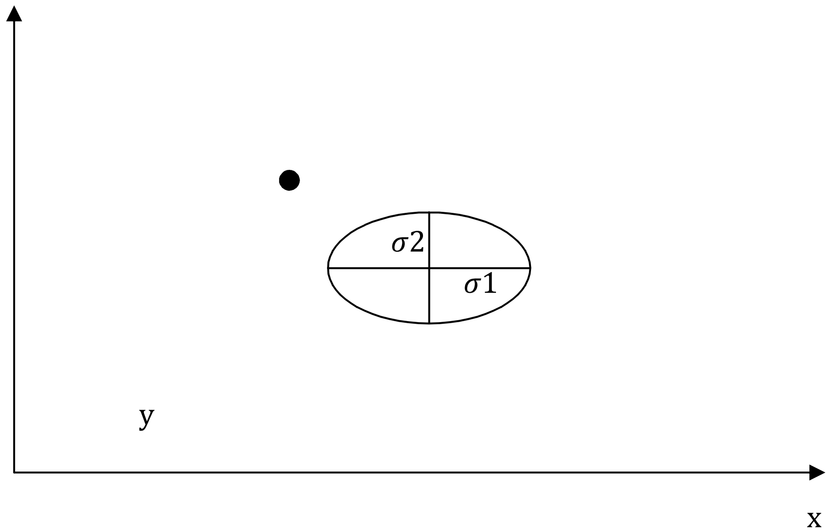


Figure 2.7 Uncorrelated Mutation With n Step Sizes

Let's assume that we have a population with n=2 and since this is an uncorrelated mutation with n step sizes; $n_{\sigma} = n = 2$, $n_{\alpha} = 0$. In Figure 2.7 the black dot represents the local maximum and the ellipse indicates the points where the offsprings can be placed with a given probability. The probability of moving along both the x-axis and y-axis <u>are not</u> the same. The probability of moving along x-axis (large effect on fitness) is larger than the probability of moving along the y-axis (small effect on fitness).

2.4.2.4.3 Correlated Mutations

In this case of mutation, the different distribution patterns are used to mutate the values of $\vec{x}$ and also the mutation has rotation angles. In the previous form of mutation explained in 2.4.2.4.2 the shape of the search space is ellipse but it was still orthogonal to the axes. This version allows the ellipse to have any orientation by rotating it with rotation matrix $\boldsymbol{C}$. That means we have different values of the strategy parameter σ for each variable of an individual and we have also rotation angles. This type of mutation can be expressed as:

$$c_i = \sigma_i^2 \tag{2-28}$$

$$c_{ij, i \neq j} = \begin{cases} 0 & , \text{ no correlations} \\ 1/2\left(\sigma_i^2 - \sigma_j^2\right)\tan\left(2\alpha_i\right) & , \text{ correlations} \end{cases} \tag{2-29}$$

$$\sigma_i{}' = \sigma_i \cdot \exp\left[\tau' \cdot N(0,1) + \tau \cdot N(0,1)\right] \tag{2-30}$$

$$\alpha_j{}' = \alpha_j + \beta \cdot \mathrm{N}(0,1) \tag{2-31}$$

$$\vec{x}' = \vec{x} + \vec{N}(\vec{0}, \boldsymbol{C}') \tag{2-32}$$

Where $n_\alpha = n \cdot \frac{n-1}{2}$.

The τ and τ' denotes the proportionality constants. They can be described as the learning rate:

$$\tau \propto 1/\sqrt{2\sqrt{n}} \tag{2-33}$$

$$\tau' \propto 1/\sqrt{2n} \tag{2-34}$$

$$\beta \approx 5° \tag{2-35}$$

Also, in this case, the σ values are required not to get close to zero, so a boundary condition is set for them, and if it they get below that level, they are set to that boundary value. We also have a boundary condition for α_j values. They should lie in the range $[-\pi, \pi.]$

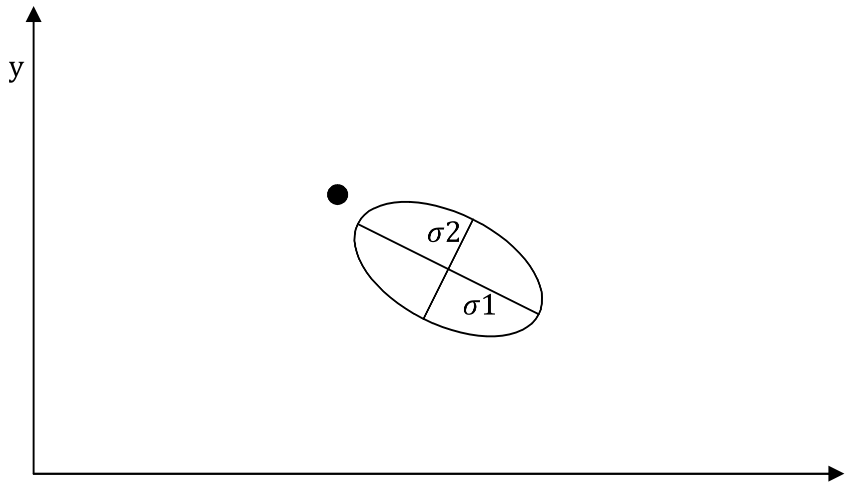


Figure 2.8 Correlated Mutations

Let's assume that we have a population with n=2 and since this is a correlated mutation; $n_{\sigma} = n = 2$, $n_{\alpha} = 1$. In Figure 2.8 the black dot represents the local maximum and the ellipse indicates the points where the offsprings can be placed with a given probability. The probability of moving along both the x-axis and y-axis <u>are not</u> the same. The probability of moving along x-axis (large effect on fitness) is larger than the probability of moving along the y-axis (small effect on fitness). Also the ellipse is not orthogonal to the axes, as in the previous cases.

In fact, the essence of mutation in ESs lies in the applications of a multiplicative log-normal distribution based modification for $\vec{\sigma}$ to ensure that standard deviations always remain positive, and of an additive normal distribution based modification for $\vec{\alpha}$ and $\vec{c}$. These particular mutation operators are motivated and justified by the argument that they permit the occurrence of small modifications more frequently than larger ones. It is crucial to highlight that the effectiveness and robustness of the mutation operator stem from the inherent self-adaptation capability of the strategy parameters. The fact that the strategy parameters are allowed to evolve together with the design variables during the course of optimization, improves convergence rate and reliability of the algorithm. For example, standard deviations attain large and small values from time-to-time to avoid entrapment in a local optimum in the former, and to accomplish a more exploitative search in the latter. Likewise, rotation angles are automatically adjusted to suitable values to determine the optimal search direction.

2.4.2.5 Selection

Selection is implemented next to determine the survivors out of parent and offspring populations. There are (μ,λ) and $(\mu+\lambda)$ type of selection strategy in ESs. In (μ,λ) selection, 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. In $(\mu+\lambda)$ selection, the selection is made among both parents and offsprings. The selected (surviving) individuals become the parents of the next generation.

The (μ,λ)-selection is preferred more because of the following reasons:

- The (μ,λ) discards all parents and is therefore in principle able to leave small local optima, so it is advantageous in the case of multimodal topologies.
- If the fitness function is not fixed, but changes in time, the $(\mu+\lambda)$ selection preserves outdated solutions, so it is not able to follow the moving optimum well.
- $(\mu+\lambda)$ selection hinders the self-adaptation mechanism with respect to strategy parameters to work effectively, because maladapted strategy parameters may survive for a relatively large number of generations

when an individual has relatively good object variables and bad strategy parameters. In that case often all its children will be bad, so with elitism, the bad strategy parameters may survive(Eiben & Smith, 1998).

Therefore, the (μ, λ)-ESs is recommended. Investigations from a particular objective function indicating a ratio of $\frac{\mu}{\lambda} = 1/7$ is optimal, concerning the accelerating effect of self-adaptation (but μ has to be chosen clearly larger than one, e.g. $\mu = 15$)(Bäck, 1996).

2.4.2.6 Termination

The algorithm terminates when a pre-assigned parameter, such as the generation number, pre-assigned time or sufficient convergence is reached, and the best individual sampled thus far is regarded to be the optimum solution.

2.4.3 ESs for Discrete Variables

As stated previously, the three extensions of ESs to solve discrete optimization problems were proposed by Cai and Thierauf (1993), Bäck and Schütz (1995), and Rudolph (1994). They all employ the general optimization routine of $(\mu + \lambda)$ and (μ, λ) variants of ESs, and differ from each other in terms of the application of mutation only. In our study a refined version of the Rudolph's approach is used, resulting in an increased performance of the approach. This particular approach will be described in latter sections.

2.4.3.1 Representation of an Individual and Initial Population

Initial population consists of μ number of parent solutions (individuals). As a usual procedure in any EA technique, a random initialization of design variable vectors is implemented for this purpose. That is, for each variable, a steel section is assigned arbitrarily from the associated discrete set. Apart from the vector of design variables $\vec{x}$, each individual comprises strategy parameters, Equation (2-36). Both strategy parameters are self-adaptive by nature, and are employed by the individual for establishing a problem-specific search scheme in an automated manner.

$$\vec{a} = \vec{a}(\vec{x}, \vec{s}) \tag{2-36}$$

In Equation (2-36) $\vec{x}$ stands for the design vector, corresponding to that component of the individual where the information related to n number of independent design variables is stored. The second component $\vec{s}$ represents the set of strategy parameters employed by the individual for establishing an automated problem-specific search mechanism in exploring the design space.

A random initialization of population is implemented for the design vectors, and the strategy parameters are assigned to appropriate values initially based on numerical experimentation.

2.4.3.2 Constraint Handling and Evaluation of Population

Constraint handling and evaluation of population of ESs having discrete variables are identical to discrete ESs. It is described in section 2.4.2.2.

2.4.3.3 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 individuals (offsprings). Recombination can be applied not only to design vectors, but also to the strategy parameters of the individuals in a variety of different schemes. In the present study a global discrete recombination operator is utilized for design variables, whereas strategy parameters are recombined using intermediate scheme. Given that $\vec{v}$ represents an arbitrary component of an individual, the recombined $\vec{v}'$ can be formulated as follows:

$$v_i' = \begin{cases} v_i^a \ o\ v_i^b, & g\ l\ o\ b\ d\ i\ s\ c\ r \\ v_i^a + (v_i^b - v_i^a)/2, & i\ n\ t\ e\ r\ m\ e \end{cases} \tag{2-37}$$

Where v^a and v^b refer to the $\vec{v}$ component of two parent individuals which are chosen randomly from the parent population, and v_i^a and v_i^b represent typical elements of $\vec{v}^b$ and $\vec{v}^a$. In global discrete recombination, v_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.

2.4.3.4 Mutation

Every offspring individual is subjected to mutation, resulting in a new set of values for the design variables ($\vec{x}$) and strategy parameters ($\vec{s}$) of the individual, Equation (2-38). This implies that not only the design information, but also the search strategy of the individual is altered during this process.

$$m\ u\left(\vec{a}(\vec{x},\vec{s})\right) = \vec{a}'(\vec{x}',\vec{s}') \tag{2-38}$$

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:

$$\vec{x}' = \vec{x} + \vec{z} \quad \textbf{(2-39)}$$

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

As far as discrete size optimum design of structures is concerned, design variables correspond to cross-sectional areas of the structural members, which are chosen from ready sections in a given profile list. To identify different sections in a profile list, each section is indicated with a separate index number between 1 and n_s, where n_s denotes the total number of ready sections in the profile list. It is essential to highlight that the application of mutation for these problems is actually performed using these indexes. That is to say, a design variable initially corresponding to x_i-th ready section of the profile list is assigned to $x_i + z_i$-th section, after Equation (2-39) is performed.

2.4.3.4.1 Mutation Approach Proposed By Rudolph

Rudolph developed an adaptive reformulation of ESs 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:

$$P(g) = \frac{1}{\Psi + 1} \cdot \left(1 - \frac{1}{\Psi + 1}\right)^g, g \in \{0,1,2, \dots, +\infty) \quad \textbf{(2-40)}$$

Where g represents a geometrically distributed integer random number, and Ψ corresponds to the mean (expectation) of this particular distribution.

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 $\Psi_{\mathrm{i}}, i \in \{1,2, \dots, n\}$ parameter, and the individual is described as follows:

$$\vec{a} = \vec{a}(\vec{x}, \vec{s}(\vec{\Psi})) \tag{2-41}$$

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 here, where the parameter p is incorporated and coupled with the original set of strategy parameters $\vec{\Psi}$ for a harmonized implementation of the mutation operator. Accordingly, in the refined form of the Rudolph's approach, an individual is described as follows:

$$\vec{a} = \vec{a}(\vec{x}, \vec{s}(\vec{p}, \vec{\Psi})) \tag{2-42}$$

where $\vec{p} = [p_1, \dots p_i, \dots p_n]$ 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. Consequently, the number of mutation probabilities (strategy parameters) employed per individual is set to one, i.e.:

$$\vec{a} = \vec{a}(\vec{x}, \vec{s}(p, \vec{\Psi})) \tag{2-43}$$

$$p' = \left(1 + \frac{1-p}{p} \cdot \exp\left(-\gamma \cdot N(0,1)\right)\right)^{-1} \tag{2-44}$$

$$\gamma = \frac{1}{\sqrt{2\sqrt{n}}} \tag{2-45}$$

In this framework, the parameter p is mutated first via Equation (2-44). Analogous to former approaches, a random number $r_i \epsilon [0,1]$ 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 a lognormal distribution based variation (Equation (2-46)), and is enforced to remain greater than 1.0 to preserve effectiveness of the mutation operator.

$$\Psi_i' = \begin{cases} \Psi_i, & \text{if } r_i > p' \\ \Psi_i \cdot \exp(\tau \cdot N_i(0,1) \geq 1, & \text{if } r_i \leq p' \end{cases} \tag{2-46}$$

In Equation (2-46), 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. 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 (2-47).

$$z_i = \begin{cases} 0, & \text{if } r_i > p' \\ g_{i,1} - g_{i,2}, & \text{if } r_i \le p' \end{cases} \tag{2-47}$$

As a final point, 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 (2-48).

$$g_{i,1}, g_{i,2} = \log(1 - r_i) / \log\left(1 - \frac{1}{1 + \Psi_i{}'}\right) \tag{2-48}$$

2.4.3.5 Selection

Selection of ESs having discrete variables is identical to the ESs having continuous variables. The selection mechanism of ESs having continuous variables is explained in section 2.4.2.5.

2.4.3.6 Termination

Termination process is same as the termination process of continuous ESs. The algorithm terminates when a pre-assigned parameter, such as the generation number, pre-assigned time or sufficient convergence is reached, and the best individual sampled thus far is regarded to be the optimum solution.

Chapter 3

DESIGN LOADS

The structures are subjected to various gravity loads in addition to lateral wind forces. The gravity loads acting on structures cover dead, live and snow loads. In the following sections, calculations of the design loads are given in detail.

3.1 Wind Loads

Wind causes significant loads on structures and these loads causes safety problems for the occupants. Hurricane winds are the largest single cause of economic and insured losses due to natural disasters, well ahead of earthquakes and floods. For example, in the United States between 1986 and 1993, hurricanes and tornadoes caused $41 billion in insured catastrophic losses, compared with $6.18 billion for all other natural hazards combined, hurricanes being the largest contributor to the losses. In Europe in 1900 alone, four winter storms caused $10 billion in insured losses, and an estimated $15 billion in economic losses (Taranath, Wind and Earthquake Resistant Buildings, 2005). Although one might think that modern buildings are more resistant, they are actually highly susceptible to wind motions because of having less weight compared to old buildings so design considerations have to change. The wind loads have to be considered more in modern buildings.

There are lots of wind load provisions that are currently used. Two main provisions are:

- Uniform Building Code (UBC) 1997.
- ASCE Minimum Design Loads For Buildings and Other Structures (ASCE 7-05)

These standards are continuously changing and adapting themselves to the evolving technology of the construction field. The resulting complexity in the determination of wind loads may be appreciated by comparing the 1973 Standard Building Code (SBC), which contained only a page and one-half of wind load requirements, to the 2002 edition of the ASCE 7, which contains 97 pages of text, commentary, figures, and tables to predict wind loads for a particular structure (Taranath, Wind and Earthquake Resistant Buildings, 2005).

Although these provisions use different methods to calculate the wind loads, they have common features. They all comprise:

- A specification of basic or reference wind speed for various locations, or zones, within a jurisdiction. Almost always a reference height of 10m in open country terrain is chosen.
- Modification factors for the effects of height and terrain type, and sometimes for change of terrain, wind direction, topography and shelter.
- Shape factors (pressure or force coefficients) for structures of various shapes.
- Some account of possible resonant dynamic effect of wind on flexible structures (Holmes, 2001).

In this thesis the ASCE 7-05 provision of calculating wind loads on buildings will be explained since this provision is used as a basis for calculating the wind loads for structural systems considered in this study.

3.1.1 Calculation of Wind Loads According to ASCE 7-05 Method 2

ASCE is the abbreviation for the American Society of Civil Engineers and the standard is for calculating the Minimum Design Loads for Buildings and Other Structures.

In ASCE 7-05 there are three different methods for calculating the wind loads for structures.

Method 1 is named as “Simplified Procedure” and in order to use this method the building must meet 7 specific requirements. Some of them are:

- It must be a simple diaphragm building as defined in section 6.2 of ASCE 7-05.
- It must have roof slopes less than 10 degree.
- The mean roof height of it must be less than or equal to 30 ft.

Method 2 is the “Analytical Procedure”. This method will be explained in more detail in the following section since it is used throughout this study. This method is applicable to structures of all size.

Method 3 is the “Wind Tunnel Procedure” and it is not an analytical model as the name implies and it is mainly used for irregular shaped buildings.

3.1.1.1 Method 2, the Analytical Procedure

In this section calculation of wind loads according to Method 2 of ASCE 7-05 will be explained.

The velocity pressure (windward face) q_z can be calculated with the Equation (3-1):

$$q_z = 0.00256 K_z K_{z\,t} K_d V^2 I \; (q_z \; i \; \imath p \;\; s \;\; a \; n \;\; V \; i \; \imath m \; \jmath h) \tag{3-1}$$

For calculating the velocity suction (leeward face), Equation (3-2) is used:

$$q_h = 0.00256 K_h K_{z\,t} K_d V^2 I \; (q_h \; i \; \imath p \;\; s \;\; a \; n \;\; V \; i \; \imath m \; \jmath h) \tag{3-2}$$

where:

K_z and K_h are the velocity pressure exposure coefficients. It takes into account changes in wind speed aboveground and the nature of the terrain (exposure categories). They are determined from Equation (3-3) and Equation (3-4):

$K_{z\,t}$ is the wind speed-up effect. It is a topographic factor that takes into account the effect of isolated hills or escarpments located in exposures B, C and D. Buildings sited on the upper half of an isolated hill or escarpment may experience significantly higher wind speeds than buildings situated on level ground. To account for these higher wind speeds, the velocity pressure exposure coefficients are multiplied by a topographic factor $K_{z\,t}$ determined from the three multipliers K_1, K_2, K_3. K_1 is related to the shape of the topographic feature and the maximum speed-up with distance upwind or downward of the rest, K_2 accounts for the reduction in speed-up with distance upwind or downward of the crest, and K_3 accounts for the reduction in speed-up with height above the local ground surface. It is determined from Figure 3.1.

K_d is the wind directionality factor. It accounts for the reduced probability of maximum winds flowing from any given direction and the reduced probability of the maximum pressure coefficient occurring for any given direction. It is determined from Figure 3.2.

V is the basic wind speed in miles per hour.

I is the importance Factor. It is a dimensionless parameter that accounts for the degree of hazard to human life and damage to property. It is determined from Table 3.2.

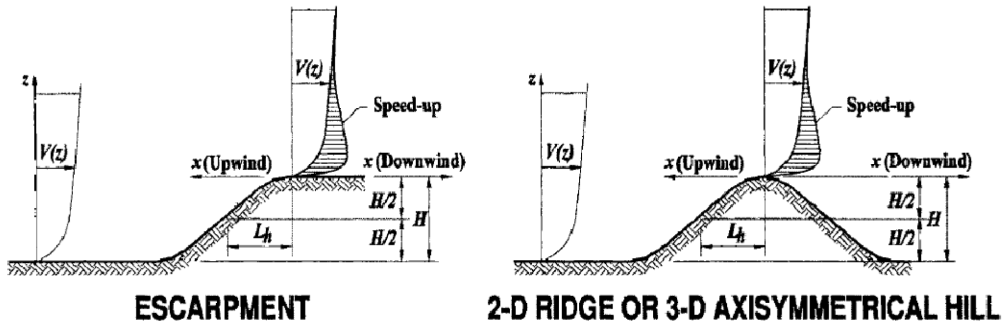
$$K_z = \begin{cases} 2.01\left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}}, & 15ft. \leq z \leq z_g \\ 2.01\left(\frac{15}{z_g}\right)^{\frac{2}{\alpha}}, & z < 15ft. \end{cases} \tag{3-3}$$

$$K_h = 2.01\left(\frac{h}{z_g}\right)^{\frac{2}{\alpha}} \tag{3-4}$$

where α is the power coefficient. It is the exponent for velocity increase in height. z_g is the gradient height above which the frictional effect of terrain becomes negligible. z is the height of the floor from the ground. h is the total height of the structure. Values of α and z_g are tabulated in Table 3.1.

Table 3.1 α and z_g values

Exposure	α	z_g (ft)
B	7.0	1200
C	9.5	900
D	11.5	700



Topographic Multipliers for Exposure C										
	K_1 Multiplier				K_2 Multiplier			K_3 Multiplier		
H/L_h	2-D Ridge	2-D Escarp.	3-D Axisym. Hill	x/L_h	2-D Escarp.	All Other Cases	z/L_h	2-D Ridge	2-D Escarp.	3-D Axisym. Hill
0.20	0.29	0.17	0.21	0.00	1.00	1.00	0.00	1.00	1.00	1.00
0.25	0.36	0.21	0.26	0.50	0.88	0.67	0.10	0.74	0.78	0.67
0.30	0.43	0.26	0.32	1.00	0.75	0.33	0.20	0.55	0.61	0.45
0.35	0.51	0.30	0.37	1.50	0.63	0.00	0.30	0.41	0.47	0.30
0.40	0.58	0.34	0.42	2.00	0.50	0.00	0.40	0.30	0.37	0.20
0.45	0.65	0.38	0.47	2.50	0.38	0.00	0.50	0.22	0.29	0.14
0.50	0.72	0.43	0.53	3.00	0.25	0.00	0.60	0.17	0.22	0.09
				3.50	0.13	0.00	0.70	0.12	0.17	0.06
				4.00	0.00	0.00	0.80	0.09	0.14	0.04
							0.90	0.07	0.11	0.03
							1.00	0.05	0.08	0.02
							1.50	0.01	0.02	0.00
							2.00	0.00	0.00	0.00

Notes:

1. For values of H/L_h, x/L_h and z/L_h other than those shown, linear interpolation is permitted.
2. For $H/L_h > 0.5$, assume $H/L_h = 0.5$ for evaluating K_1 and substitute 2H for L_h for evaluating K_2 and K_3.
3. Multipliers are based on the assumption that wind approaches the hill or escarpment along the direction of maximum slope.
4. Notation:
   H: Height of hill or escarpment relative to the upwind terrain, in feet (meters).
   L_h: Distance upwind of crest to where the difference in ground elevation is half the height of hill or escarpment, in feet (meters).
   K_1: Factor to account for shape of topographic feature and maximum speed-up effect.
   K_2: Factor to account for reduction in speed-up with distance upwind or downwind of crest.
   K_3: Factor to account for reduction in speed-up with height above local terrain.
   x: Distance (upwind or downwind) from the crest to the building site, in feet (meters).
   z: Height above local ground level, in feet (meters).
   μ: Horizontal attenuation factor.
   γ: Height attenuation factor.

Figure 3.1 Wind Speed-Up Effect

Equations:

$$K_{zt} = (1 + K_1 K_2 K_3)^2$$

K_1 determined from table below

$$K_2 = (1 - \frac{|x|}{\mu L_h})$$

$$K_3 = e^{-\gamma z / L_h}$$

Parameters for Speed-Up Over Hills and Escarpments						
Hill Shape	$K_1/(H/L_h)$			γ	μ	
	Exposure				Upwind of Crest	Downwind of Crest
	B	C	D			
2-dimensional ridges (or valleys with negative H in $K_1/(H/L_h)$)	1.30	1.45	1.55	3	1.5	1.5
2-dimensional escarpments	0.75	0.85	0.95	2.5	1.5	4
3-dimensional axisym. hill	0.95	1.05	1.15	4	1.5	1.5

Figure 3.1 (continued)

Structure Type	Directionality Factor K_d*
Buildings	
Main Wind Force Resisting System	0.85
Components and Cladding	0.85
Arched Roofs	0.85
Chimneys, Tanks, and Similar Structures	
Square	
Hexagonal	0.90
Round	0.95
	0.95
Solid Signs	0.85
Open Signs and Lattice Framework	0.85
Trussed Towers	
Triangular, square, rectangular	0.85
All other cross sections	0.95

Figure 3.2 Wind Directionality Factor

Table 3.2 Importance Factor

Category	Non-Hurricane Prone Regions and Hurricane Prone Regions with V=85-100 mph and Alaska	Hurricane Prone Regions with V>100 mph
I	0.87	0.77
II	1.00	1.00
III	1.15	1.15
IV	1.15	1.15

After calculating the velocity pressure, the design pressures for windward and leeward faces are calculated from Equation (3-5) and Equation (3-6):

$$P_W = q_z C_{pW} G \ (Pw \text{ in } lb/ft2) \quad \textbf{(3-5)}$$

$$P_l = q_h C_{pl} G \ (Pl \text{ in } lb/ft2) \quad \textbf{(3-6)}$$

Where:

C_{pW} and C_{pl} are the external pressure coefficients. It is determined from Figure 3.3.

G: Gust effect factor. It accounts for additional dynamic amplification of loading in the along-wind direction due to wind turbulence and structure interaction. It does not include allowances for across-wind loading effects, vortex shedding, instability due to galloping or flutter, or dynamic torsional effects. Buildings susceptible to these effects should be designed using wind tunnel results.

It can be taken as 0.85 for rigid structures (defined as the structures that have natural frequency of vibration greater than 1 Hz).

Wall Pressure Coefficients, C_p			
Surface	**L/B**	C_p	**Use With**
Windward Wall	All values	0.8	q_z
Leeward Wall	0-1	-0.5	q_h
	2	-0.3	
	≥4	-0.2	
Side Wall	All values	-0.7	q_h

Roof Pressure Coefficients, C_p, for use with q_h

Wind Direction	Windward									Leeward		
	Angle, θ (degrees)									Angle, θ (degrees)		
	h/L	10	15	20	25	30	35	45	≥60#	10	15	≥20
Normal to ridge for θ ≥ 10°	≤0.25	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.3	-0.2 0.3	0.0* 0.4	0.4	0.01 θ	-0.3	-0.5	-0.6
	0.5	-0.9 -0.18	-0.7 -0.18	-0.4 0.0*	-0.3 0.2	-0.2 0.2	-0.2 0.3	0.0* 0.4	0.01 θ	-0.5	-0.5	-0.6
	≥1.0	-1.3** -0.18	-1.0 -0.18	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.2	0.0* 0.3	0.01 θ	-0.7	-0.6	-0.6

Wind Direction	h/L	Horiz distance from windward edge	C_p
Normal to ridge for θ < 10 and Parallel to ridge for all θ	≤ 0.5	0 to h/2	-0.9, -0.18
		h/2 to h	-0.9, -0.18
		h to 2 h	-0.5, -0.18
		> 2h	-0.3, -0.18
	≥ 1.0	0 to h/2	-1.3**, -0.18
		> h/2	-0.7, -0.18

*Value is provided for interpolation purposes.

**Value can be reduced linearly with area over which it is applicable as follows

Area (sq ft)	Reduction Factor
≤ 100 (9.3 sq m)	1.0
200 (23.2 sq m)	0.9
≥ 1000 (92.9 sq m)	0.8

Notes:

1. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
2. Linear interpolation is permitted for values of L/B, h/L and θ other than shown. Interpolation shall only be carried out between values of the same sign. Where no value of the same sign is given, assume 0.0 for interpolation purposes.
3. Where two values of C_p are listed, this indicates that the windward roof slope is subjected to either positive or negative pressures and the roof structure shall be designed for both conditions. Interpolation for intermediate ratios of h/L in this case shall only be carried out between C_p values of like sign.
4. For monoslope roofs, entire roof surface is either a windward or leeward surface.
5. For flexible buildings use appropriate G_f as determined by Section 6.5.8.
6. Refer to Figure 6-7 for domes and Figure 6-8 for arched roofs.
7. Notation:
   B: Horizontal dimension of building, in feet (meter), measured normal to wind direction.
   L: Horizontal dimension of building, in feet (meter), measured parallel to wind direction.
   h: Mean roof height in feet (meters), except that eave height shall be used for θ ≤ 10 degrees.
   z: Height above ground, in feet (meters).
   G: Gust effect factor.
   q_z, q_h: Velocity pressure, in pounds per square foot (N/m^2), evaluated at respective height.
   θ: Angle of plane of roof from horizontal, in degrees.
8. For mansard roofs, the top horizontal surface and leeward inclined surface shall be treated as leeward surfaces from the table.
9. Except for MWFRS's at the roof consisting of moment resisting frames, the total horizontal shear shall not be less than that determined by neglecting wind forces on roof surfaces.

#For roof slopes greater than 80°, use C_p = 0.8

Figure 3.3 External Pressure Coefficient

3.2 Dead Loads

Dead loads consists of the weight of all materials of construction incorporated in the building including, walls, floors, roofs, ceilings, stairways, built-in partitions, finishes, cladding and other similarly incorporated architectural and structural items (ASCE, 2006).

Equivalent dead load on short side of the frame can be calculated by the following equation:

$$p\ \iota_s = \frac{1}{3} l_s D_d \tag{3-7}$$

Equivalent dead load on long side of the frame can be calculated according to the equation:

$$p\ \iota_l = \frac{1}{3} l_s D_d \left(3/2 - \frac{1}{2m^2}\right) \tag{3-8}$$

$$m = l_l / l_s \tag{3-9}$$

where $p\ d_s$ and $p\ d_l$ are the equivalent dead loads on short side and long side of the frame respectively. l_s and l_l is the length of the short side and long side of the frame respectively. D_d is the design dead load.

3.3 Live Loads

Equivalent live load on short side of the frame can be calculated by the following equation:

$$p\ _s = \frac{1}{3} l_s D_l \tag{3-10}$$

Equivalent live load on long side of the frame can be calculated according to the equation:

$$p^{l}_{l} = \frac{1}{3} l_s D_l \left(3/2 - \frac{1}{2m^2}\right) \quad \textbf{(3-11)}$$

$$m = l_l / l_s \quad \textbf{(3-12)}$$

where p^{l}_{s} and p^{l}_{l} are the equivalent live loads on short side and long side of the frame respectively. D_l is the design live load.

3.4 Snow Load

Calculation of snow loads are similar to the calculation of dead and live loads:

$$p^{s}_{s} = \frac{1}{3} l_s D_s \quad \textbf{(3-13)}$$

$$p^{s}_{l} = \frac{1}{3} l_s D_s \left(3/2 - \frac{1}{2m^2}\right) \quad \textbf{(3-14)}$$

$$m = l_l / l_s \quad \textbf{(3-15)}$$

where p^{s}_{s} and p^{s}_{l} are the equivalent snow loads on short side and long side of the frame respectively. l_s is the length of the short side of the frame. D_s is the design snow load.

Numerical values of the deign loads are given in Chapter 6.

Chapter 4

BRACING SYSTEMS

A steel frame can be strengthened in various types to resist lateral forces. These systems are, moment-resisting beam-column connections (model-A) (Figure 4.1), braced frames with moment-resisting connections (model-B to model-G) (Figure 4.2 to Figure 4.7), braced frames with pin-jointed connections (model-H to model-J) (Figure 4.8 to Figure 4.10) and braced frames with both pin-jointed and moment-resisting connections (model-K to model-M) (Figure 4.11 to Figure 4.13). An engineer can select one of the lateral load resisting systems. In steel buildings the most widely used method of constructing lateral load resisting system is using braced frames. Hence, the main concern is to select the appropriate bracing model and to decide the suitable connection type.

Bracing systems are used in structures in order to resist lateral forces. Diagonal structural members are inserted into the rectangular areas so that triangulation is formed. These systems help the structure to reduce the bending of columns and beams and the stiffness of the system is increased.

There are lots of advantages of the bracing systems so that they are widely used. These are:

- Braced frames are applicable to all kind of structures like bridges, aircrafts, cranes, buildings and electrical transmission towers.

- Braced frames are easy to fabricate and construct. No lots of knowledge or skills are needed.
- If the bolted connections are used, there is no deformation problem at the connections.
- The design of the braced systems is simple because the system can be separated in two parts such that the vertical loads resisting parts and horizontal loads resisting parts.

Simple braced non-sway frames may be considered as cantilevered vertical trusses resisting lateral loads primarily through the axial stiffness of columns and braces. The columns act as the cords in resisting the overturning moment, with tension in the windward column and the compression in the leeward column. The diagonals work as the web members resisting the horizontal shear in axial compression or tension, depending on the direction of inclination. The beams act axially, when the system is a fully triangulated truss. They undergo bending only when the braces are eccentrically connected to them. Because the lateral loads are reversible, braces are subjected to both compression and tension; they are most often designed for the more stringent case of compression (Taranath, Wind and Earthquake Resistant Buildings, 2005).

The building, which is braced, displace very little under horizontal forces so that the horizontal displacement of them may be neglected and the building may be classified as non-sway building.

Braced-bays are located such that they have minimum impact on the structural layout, but taking into account the manner in which the frame is to be erected, the distribution of horizontal forces and the location of any movement joints in the structure.

Braced-bay systems comprise diagonal, cross, 'K' and eccentric bracing arrangements. The advantage of triangulated systems is that the bracing elements are subjected only to tension or tension and compression in the absence of bending moments. Consequently, the members are relatively light providing a very stiff overall structural response. In the case of eccentric bracing, the system relies, in part, on flexure of the horizontal beam elements. This particular arrangement provides a more flexible overall response which is most effective under seismic loading conditions.

Where a single diagonal (as opposed to 'cross') brace is used, it must be capable of resisting both tensile and compressive axial forces to allow for the alternating direction of wind load. Under these conditions, it is recommended that the bracing member has a minimum slenderness ratio of 250 to prevent the self-weight deflection of the brace limiting its compressive resistance.

Although many different section shapes can be used as compression braces, a circular hollow section is the most efficient structurally. It should be noted that, in addition, hollow sections offer a greater resistance to corrosion and can be more aesthetically pleasing than open sections.

4.1 Types of Braces

There are lots of brace types; the ones that are used in this study can be seen from Figure 4.2 to Figure 4.13.

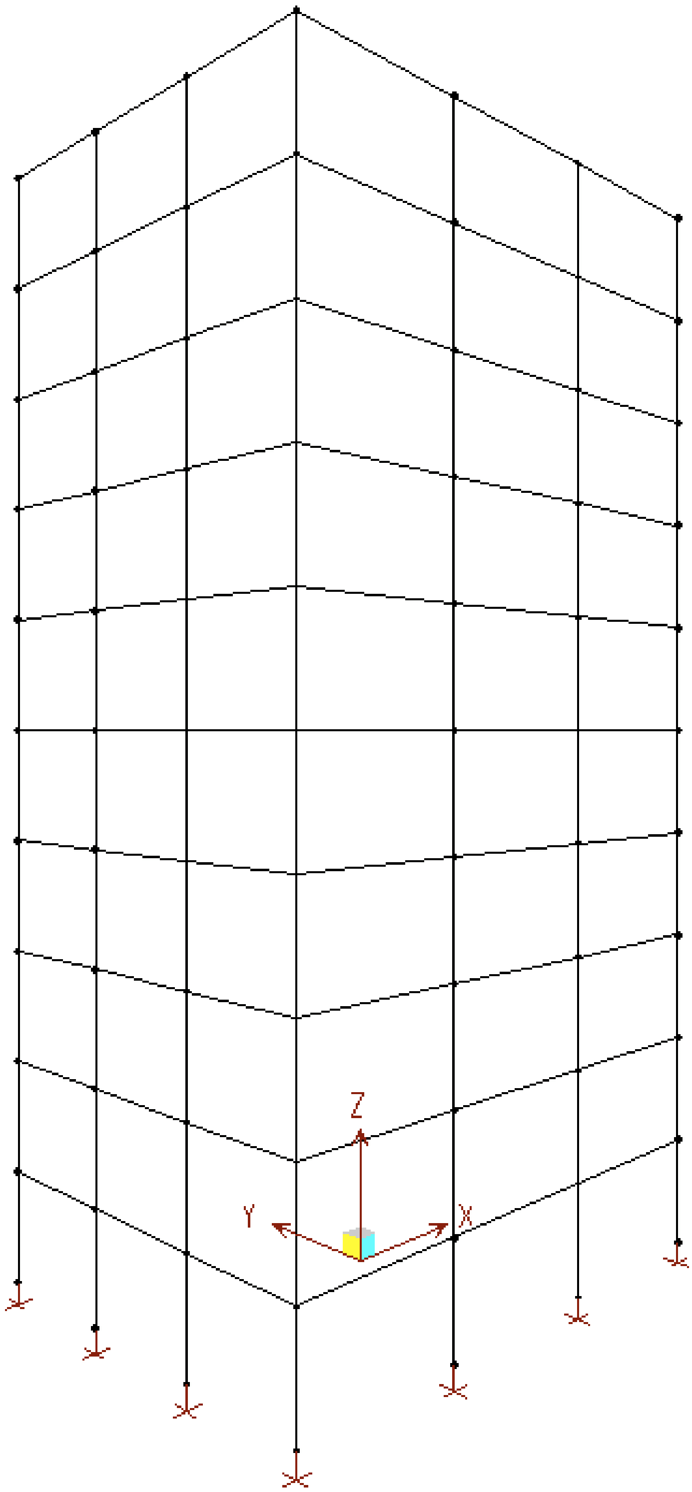


Figure 4.1 Model-A (No-Bracing)

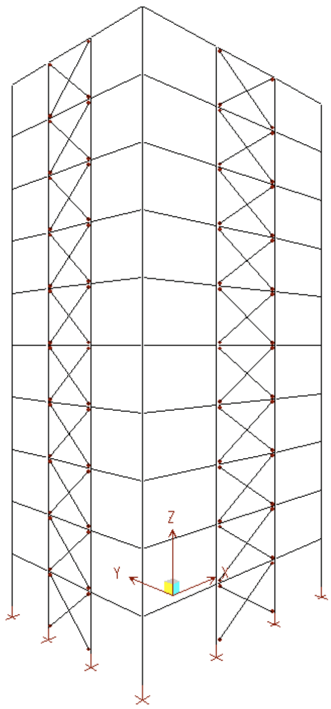


Figure 4.2 Model-B

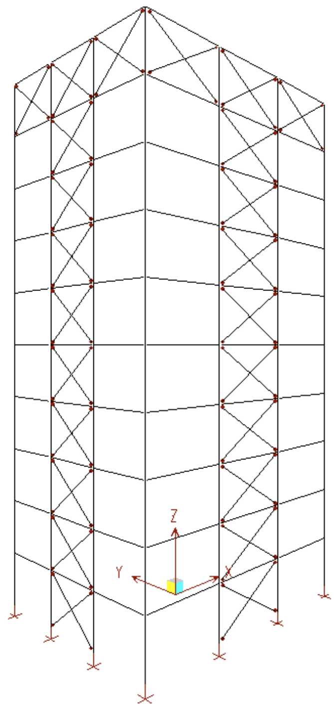


Figure 4.3 Model-C

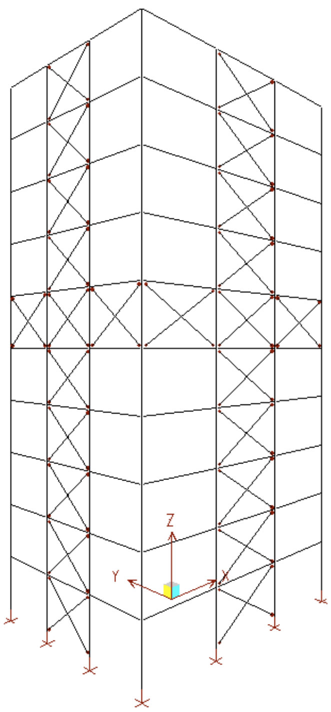


Figure 4.4 Model-D

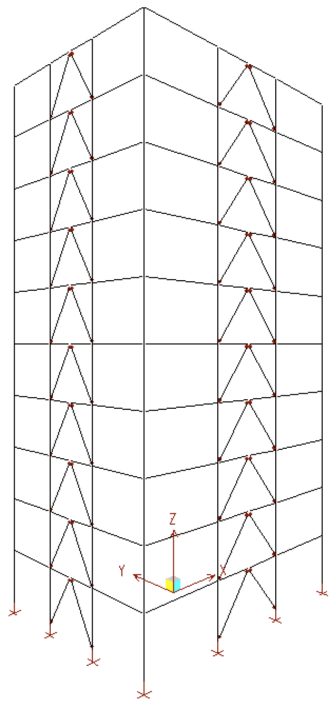


Figure 4.5 Model-E

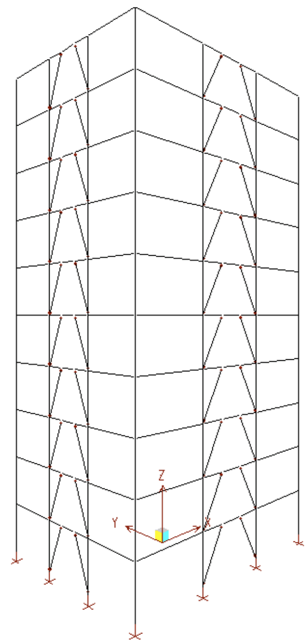


Figure 4.6 Model-F

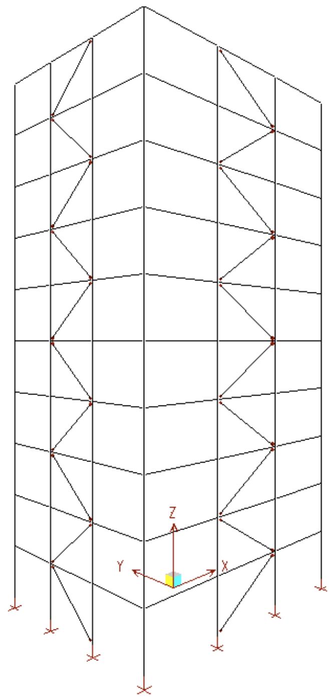


Figure 4.7 Model-G

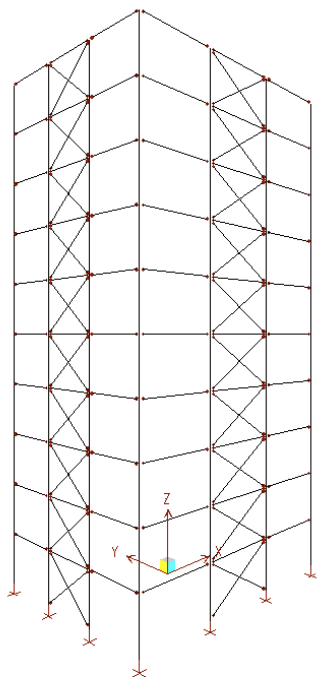


Figure 4.8 Model-H

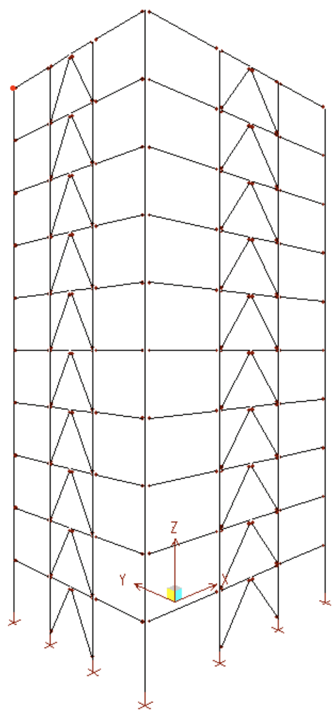


Figure 4.9 Model-I

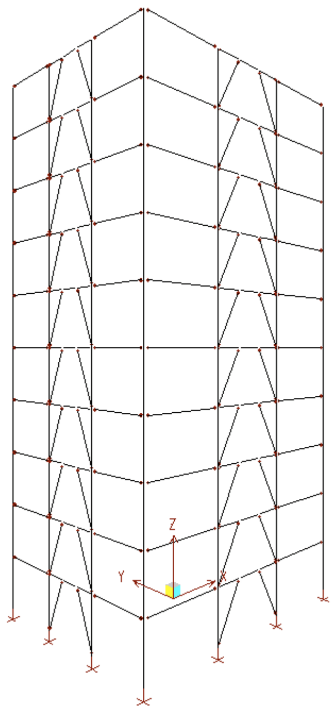


Figure 4.10 Model-J

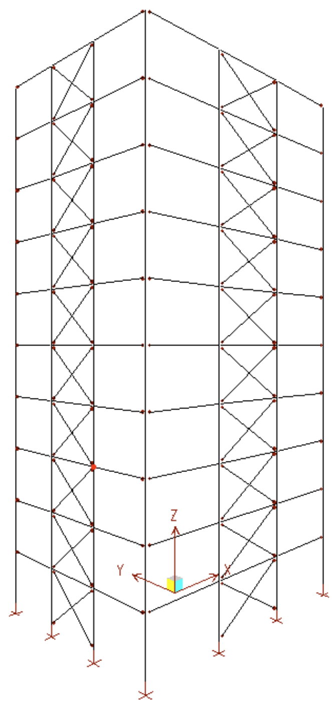


Figure 4.11 Model-K

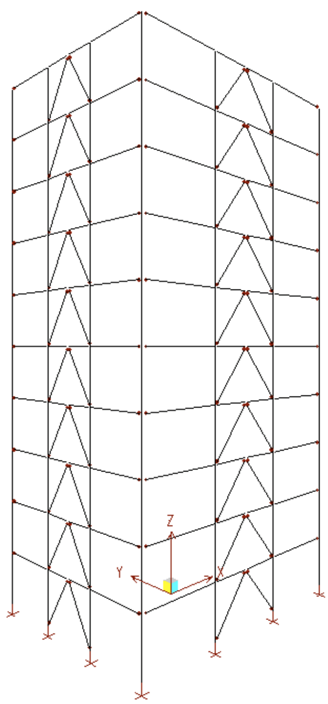


Figure 4.12 Model-L

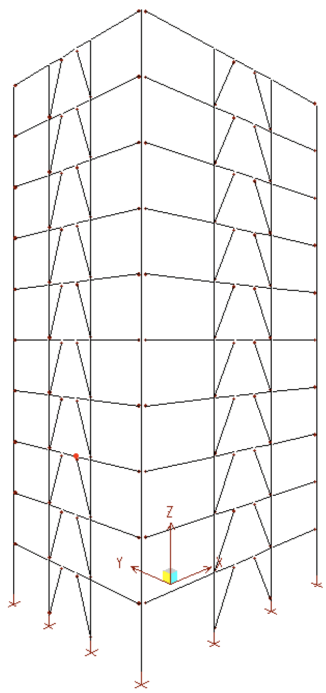


Figure 4.13 Model-M

- Model A: No bracing, all column and beam connections are rigid.
- Model B: Middle span is supported by X-type framework and all beam and column connections are rigid.
- Model C: Middle span is supported by X-type framework and all beam and column connections are rigid. Moreover top floor is supported by X-type framework.
- Model D: Middle span is supported by X-type framework and all beam and column connections are rigid. Moreover middle floor is supported by X-type framework.
- Model E: Middle span is supported by K-type framework and all beam and column connections are rigid.
- Model F: Middle span is supported by Knee-type framework and all beam and column connections are rigid.
- Model G: Middle span is supported by Z-type framework and all beam and column connections are rigid.
- Model H: Middle span is supported by X-type framework and all beam and column connections are pin-jointed.
- Model I: Middle span is supported by K-type framework and all beam and column connections are pin-jointed.
- Model J: Middle span is supported by Knee-type framework and all beam and column connections are pin-jointed.
- Model K: Middle span is supported by X-type framework and only outer most beam-column connections are joints.
- Model L: Middle span is supported by K-type framework and only outer most beam-column connections are pin-jointed.
- Model M: Middle span is supported by Knee-type framework and only outer most beam-column connections are pin-jointed.,

Chapter 5

COST ANALYSIS OF STRUCTURES

Realistic structural design optimization should consider real structural properties, multiple load cases, and constraints representing all ultimate and serviceability limit state design rules. In many studies, structure having the least design weight is considered to be the most economic one. However, this is only an assumption that the minimal weight could lead to reasonable low overall costs.

Structural costs of the models in present study are found according to the formulas given in Pavlovic, Krajnc, & Beg (2004).

Cost of a structure is mainly composed of five elements. These are total material cost of the elements (C_{el}), total material and manufacturing cost of joints (C_j), total transportation costs (C_t), total erection costs (C_{er}) and extra costs (C_{ex}). Cost of the structure (C_{ST}) can be formulated as:

$$C_{ST} = C_{el} + C_j + C_{er} + C_t + C_{ex} \tag{5-1}$$

5.1 Cost of the Elements

Steel costs for the elements depend on the mass of all structural elements and total cost of elements (C_{el}) can be calculated as:

$$C_{el} = k_m m_{el} \tag{5-2}$$

where $k_m = 0.40$ is the unit price of steel in $euro/kg$, m_{el} is the mass of all elements in the structure in kg.

5.2 Cost of the Joints

The assessment of the costs for joints is more complex and contains the sum of different process costs. The cost of joints (C_j) can be calculated as:

$$C_j = C_{jm} + C_{jn} \tag{5-3}$$

$$C_{jm} = C_{wm} + C_{bm} + C_{sm} + C_{pm} \tag{5-4}$$

$$C_{jn} = C_{wn} + C_{hn} \tag{5-5}$$

Where:

- C_{jm} is the total material cost of joints in $euro$, C_{jn} is the total manufacturing cost of joints in $euro$.
- C_{wm}, C_{bm}, C_{sm}, C_{pm} are cost of welding material, cost of bolting material, cost of stiffener material and cost of end plate material respectively.
- C_{wn} is the total manufacturing cost of welding in $euro$ and C_{hn} is the total manufacturing cost of hole forming in $euro$.

5.2.1 Material Cost of Joints

$$C_{j\,n} = C_{w\,n} + C_{b\,n} + C_{s\,n} + C_{p\,n} \tag{5-6}$$

$$C_{w\,n} = k_{w\,n} M_w L_w \tag{5-7}$$

$$C_{b\,n} = k_{b\,n} n_b \tag{5-8}$$

$$C_{s\,n} = k_s m_{s\,n} \tag{5-9}$$

$$C_{p\,n} = k_s m_{p\,n} \tag{5-10}$$

where:

- $k_{w\,m} = 1.40$ is the unit price of welding material in $euro/kg$, $M_w = 1.33a_w^2 + 0.19a_w - 0.02$ is the material consumption for welding in kg/m, L_w is the total length of welding in m, a_w is the welding size in cm and during this study it is taken equal to the plate thickness.
- $k_{b\,m} = 3.076d^2 - 7.373d + 4.62$ is the unit price of bolt in $euro/bolt$, n_b is the total number of bolts and d is the bolt diameter in cm.
- $k_s = 0.40$ is the unit price of steel in $euro/kg$, $m_{s\,m}$ is the total mass of stiffener material in kg.
- $m_{p\,m}$ is the total mass of plate material in kg.
- The cost for the middle connection of the x-type bracing members with each other is neglected for all models comprising x-type bracing.

5.2.2 Manufacturing Cost of Joints

$$C_{j} = C_{w} + C_{hn} \tag{5-11}$$

$$C_{w} = k_{w}(f_{w}T_{w}L_{w} + T_{wn}) \tag{5-12}$$

$$C_{hn}=k_{hn}(n_{h}(t_{p} + t_{p})/V_{h} + T_{hn}) \tag{5-13}$$

where:

- k_{wn}= 0.123 is the unit price of welding manufacturing in $euro/min$ f_{wn}= 1.40 is a factor that increases labor. T_{wn} is the operation time per unit length of weld in min/m and it is calculated from $(17.26a_w^2 + 2.9a_w + 1.82)$, where a_w is the weld size in cm. L_w is the welding length in m. T_{wne}= 0.3 is the any additional welding time in min
- k_{hn}= 0.323 is the unit price of hole forming in $euro/min$. n_h is the number of holes. t_p is the thickness of drilled plate in cm. t_{pe} is the additional drilling path in cm. V_h is the drilling speed in cm/min and it is calculated from $(0.763d^2 - 5.720d + 20.96)$, where d is the hole diameter in cm. T_{hne}= 11.9 is the additional time for preparation etc in min

5.3 Cost of Transportation

Since it is a difficult task to identify the transportation costs, cost estimation of transportation refers to the total mass of elements. We have to be aware that it is only an approximation. Therefore cost of transportation (C_t) can be calculated as:

$$C_t = k_t m_{st} \tag{5-14}$$

Where $k_t = 0.022$ is the unit price of transportation in $euro/kg$, m_{st} is the total mass of the structure in kg

5.4 Cost of Erection

Similar to the cost of transportation, cost of erection is a function of the mass of the structure. Cost of erection (C_{er}) can be calculated as:

$$C_{er} = k_{er} T_{er} m_{st} \tag{5-15}$$

where $k_{er} = 120.20$ is the unit price of man-hour considering the man and machine power and it is in $euro/h$, $T_{er} = 0.0014$ is the man-hour per unit quantity of the steel in $hour/kg$.

5.5 Extra Costs

In a building there are lots of extra costs that have to be taken into account. These are painting, flange aligning, surface preparation, cutting and welding of the elements. Since the models that are studied are very similar, for simplicity, the other costs are taken as a function of the mass of the structure. Extra costs are taken equal to 0.184 $euro/kg$ for all of the models. This value is decided according to the sample problem solved in Pavlovic, Krajnc, & Beg, (2004).

Chapter 6

TEST STRUCTURES

Three different structures with 10, 20 and 30 stories are considered and for every structure all of the prescribed bracing models are applied. Therefore 39 different models with practical design considerations are studied to compare performances of different bracing models and frame types. They are designed for minimum weight considering cross-sections of the members being the design variables. The costs of the models are calculated after the designs are completed regarding the minimum weight. For all design examples, the following material properties of the steel are used: modulus of elasticity (E) = 29,000 ksi (203,893.6 MPa) and yield stress (F_y) = 36 ksi (253.1 MPa).

It is necessary to prepare a design pool, where a set of steel sections selected from available profile lists in the design code are collected and sorted in descending order. The sequence numbers assigned to these sections vary between 1 to total number of sections included in the design pool. This sequence number is treated as design variable. For example, if there are 272 steel sections in the design pool and 35 member groups in the frame to be designed then the ES technique selects randomly an integer number which has a value between 1 and 272 for each member group. Once this selection is carried out for each group, the cross sectional properties of each steel section become available from the design pool. The structure is then analyzed under the external loads with these sections to find out whether its response is within the limitations imposed by the design code or not. 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.

The rigid beam-column connections are designed according to the model shown in Figure 6.1. The stiffeners are welded along three edges with fillet welds and the end plate is attached to a beam with butt welds, with weld size equal to the thickness of the beam plates. There are 8 bolts at the plate.

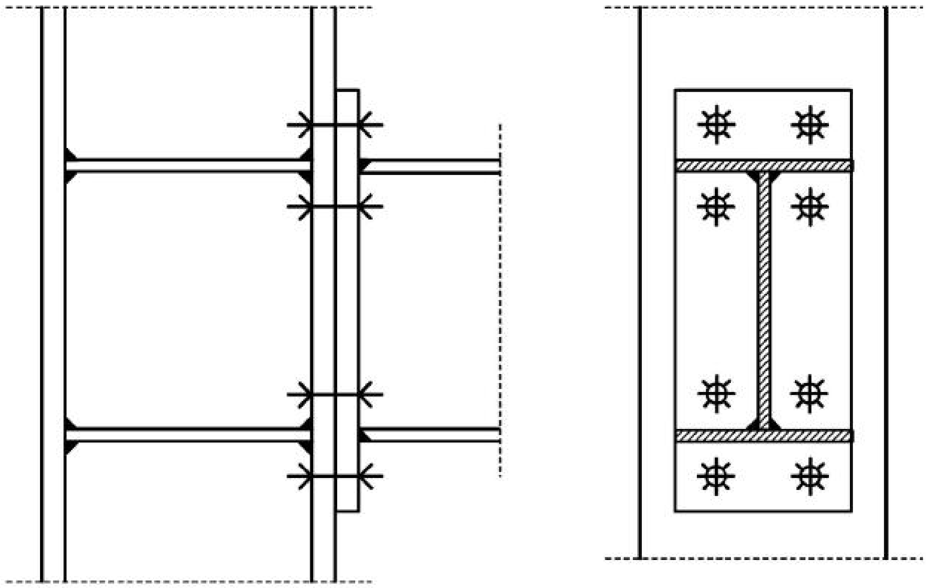


Figure 6.1 Rigid beam-column connection

The pin-jointed beam-column connections are designed according to the drawing given in Figure 6.2.

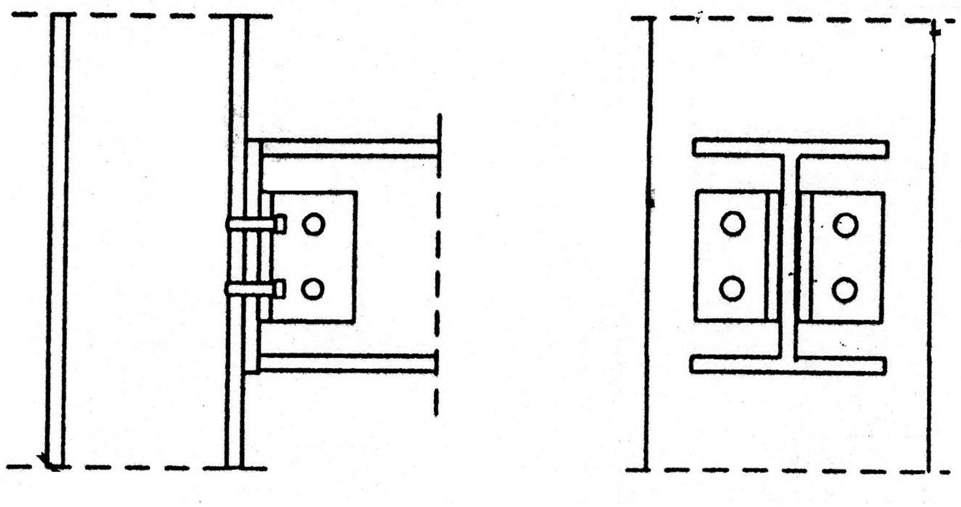


Figure 6.2 Pin-jointed beam-column connection

There are three joint types; corner joints, inner joints and side joints. These joints are illustrated in Figure 6.3. Detailed cost calculations of joints are carried out such that three joints are taken for each of 10, 20 and 30 storey structures. The shear force acting on the joints are taken as the average shear force on the structure. The costs are calculated according to formulas given in Chapter 5. The results are given in Table 6.1.

The plan dimensions of the structures are identical and it is shown in Figure 6.4. The structures are 3x3 bay frames. X-bay and y-bay dimensions are 20ft and 15ft respectively. The story height of the frames is 12ft.

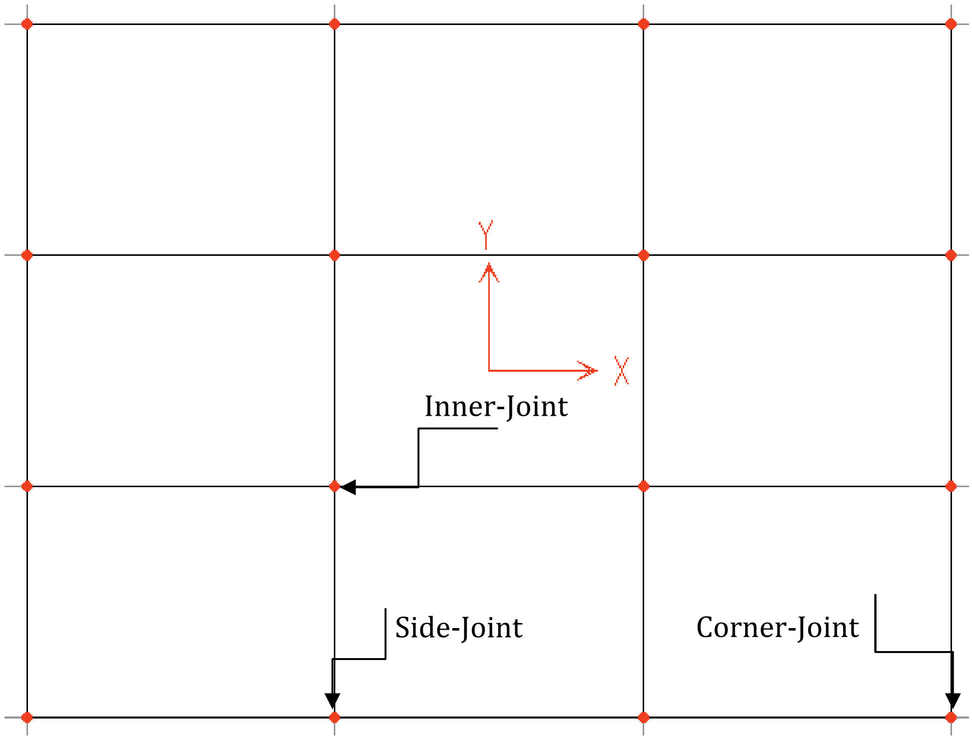


Figure 6.3 Joint Placement

Table 6.1 Joint Costs

	Joint Cost (Euro)					
	Inner Joint		Side Joint		Corner Joint	
Connection Type	Fixed	Pinned	Fixed	Pinned	Fixed	Pinned
10 Storey	68.11	10.87	101.38	14.81	152.79	18.78
20 Storey	179.84	15.90	318.06	21.88	362.77	29.49
30 Storey	323.41	44.91	462.02	64.23	602.44	84.36

By employing the symmetry of the structure and fabrication requirements of structural members, the elements are grouped such that there are 35, 70 and 105 member groups (independent size variables) for 10, 20 and 30 storey frames respectively. The member groupings are made such that:

- Inner beams at every two floors,
- Outer beams at every two floors,
- Corner columns at every two floors,
- x-z outer columns at every two floors,
- y-z outer columns at every two floors,
- Inner columns at every two floors,
- Bracings at every two floors,

have the same cross-section.

The gravity and wind forces are combined under two loading conditions. In the first loading condition, the gravity loads are applied with the wind loads acting along x-axis (i.e., 1.0GL + 1.0WL-x), whereas in the second one they are applied with the wind forces acting along y-axis (i.e., 1.0GL + 1.0WL-y).

The solution algorithm presented in Chapter 2 is computerized in optimization software (OPTSTEEL) that is compiled in Borland Delphi source code. 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.

In the following sections, numerical values of design loads are given which are calculated according to the methods given in Chapter 3.

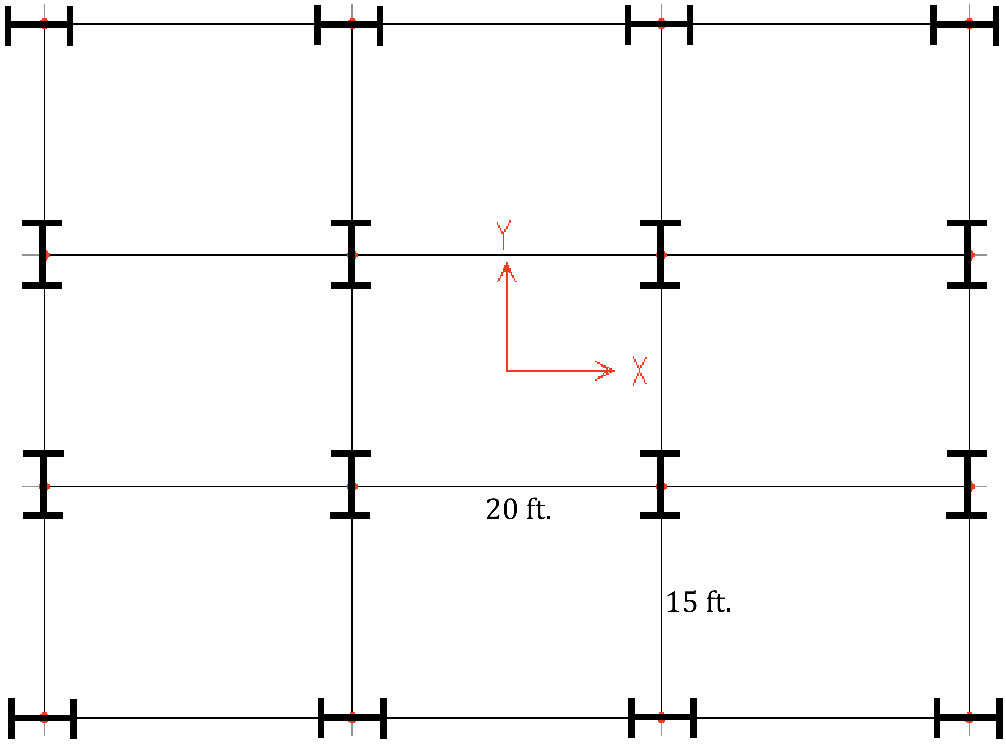


Figure 6.4 Typical plan view of the models

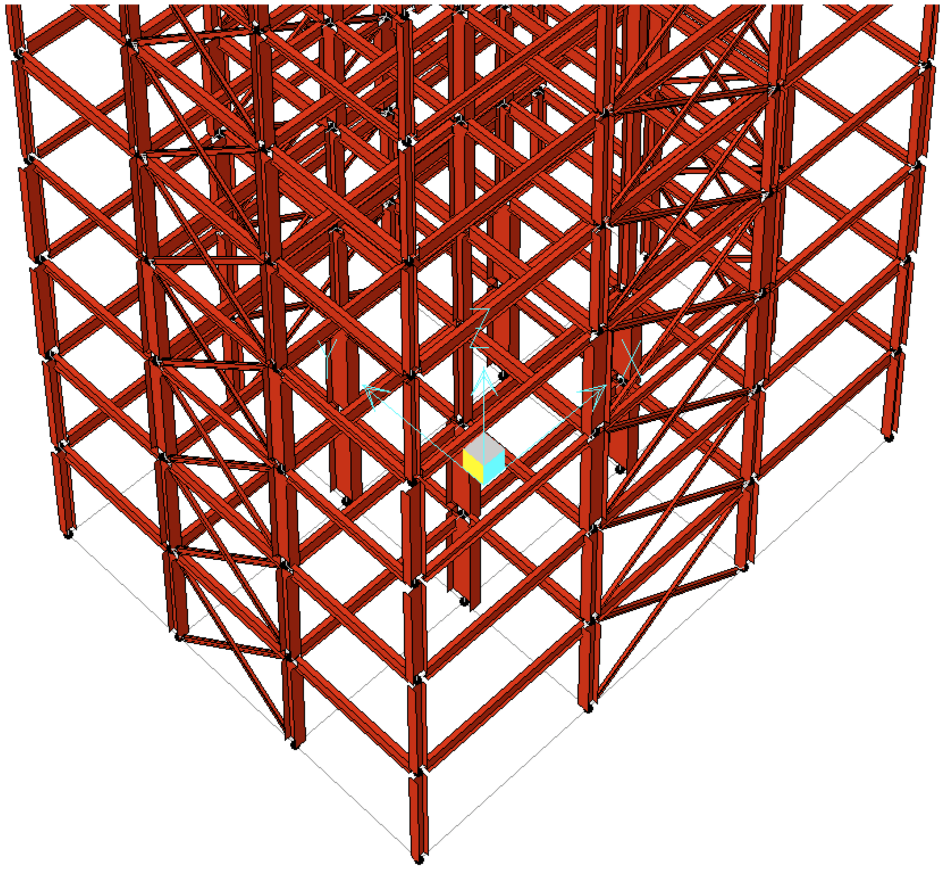


Figure 6.5 3-D view of the model-B

6.1 Calculation of Gravity Loads

All structures are subjected to various gravity loads in addition to lateral wind forces. 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 60.13 lb/ft^2 (D_d) and to design live load of 50 lb/ft^2 (D_l). The roof is subjected to a design dead load of 60.13 lb/ft^2 plus design snow load of 15.75 lb/ft^2. The gravity loads on structures are calculated as explained in Sections 3.2, 3.3 and 3.4. The resulting gravity loads on the outer and inner beams of the roof and floors are listed in Table 6.2. The calculated gravity loads are applied as uniformly distributed loads on the beams using distribution formulas developed for slabs.

Table 6.2 Gravity Loading Of the Frames

BEAM TYPE	Outer Span Beams (lb/ft)	Inner Span Beams (lb/ft)
Long Span Roof Beams	462.36	924.84
Long Span Floor Beams	671.16	1342.2
Short Span Roof Beams	379.44	758.76
Short Span Floor Beams	550.68	1101.36

6.2 Calculation of Wind Loads

The wind loads on structures are calculated according to method 2 of ASCE 7 which is explained in section 3.1.1.1. They are applied as uniformly distributed lateral loads on the external beams of the frame located at windward and leeward facades at every floor level. The basic wind speed is taken as 125 mph (200 kph). Assuming that the building is located in a flat terrain and with exposure category B, the following values are used for these parameters: K_{zt}=1, K_d=0.85, I=1, G=0.85, C_{pw}=0.8, C_{pl}=-0.44 for winds along x-direction and Cpl=-0.5 for wind along y-direction. The calculated wind loads at every floor level are presented in Table 6.3 to Table 6.8.

Table 6.3 Wind loads for x-direction winds (30-Storey Models)

Floor	z	K_z	Distributed Windward Force (lb/ft)	Distributed Leeward Force (lb/ft)
1	12	0.57	159.45	217.44
2	24	0.66	182.37	217.44
3	36	0.74	204.77	217.44
4	48	0.80	222.31	217.44
5	60	0.85	236.94	217.44
6	72	0.90	249.61	217.44
7	84	0.94	260.85	217.44
8	96	0.98	271.00	217.44
9	108	1.01	280.27	217.44
10	120	1.04	288.84	217.44
11	132	1.07	296.81	217.44
12	144	1.10	304.28	217.44
13	156	1.12	311.32	217.44
14	168	1.15	317.98	217.44
15	180	1.17	324.31	217.44
16	192	1.19	330.35	217.44
17	204	1.21	336.12	217.44
18	216	1.23	341.65	217.44
19	228	1.25	346.97	217.44
20	240	1.27	352.09	217.44
21	252	1.29	357.04	217.44
22	264	1.30	361.81	217.44
23	276	1.32	366.44	217.44
24	288	1.34	370.92	217.44
25	300	1.35	375.27	217.44
26	312	1.37	379.50	217.44
27	324	1.38	383.62	217.44
28	336	1.40	387.62	217.44
29	348	1.41	391.53	217.44
30	360	1.42	197.67	108.72
TOTAL			9,189.71	6,414.48

Table 6.4 Wind loads for x-direction winds (20-Storey Models)

Floor	z	K_z	Distributed Windward Force (lb/ft)	Distributed Leeward Force (lb/ft)
1	12	0.57	159.45	193.65
2	24	0.66	182.37	193.65
3	36	0.74	204.77	193.65
4	48	0.80	222.31	193.65
5	60	0.85	236.94	193.65
6	72	0.90	249.61	193.65
7	84	0.94	260.85	193.65
8	96	0.98	271.00	193.65
9	108	1.01	280.27	193.65
10	120	1.04	288.84	193.65
11	132	1.07	296.81	193.65
12	144	1.10	304.28	193.65
13	156	1.12	311.32	193.65
14	168	1.15	317.98	193.65
15	180	1.17	324.31	193.65
16	192	1.19	330.35	193.65
17	204	1.21	336.12	193.65
18	216	1.23	341.65	193.65
19	228	1.25	346.97	193.65
20	240	1.27	176.05	96.83
TOTAL			5,442.25	3,776.18

Table 6.5 Wind loads for x-direction winds (10-Storey Models)

Floor	z	K_z	Distributed Windward Force (lb/ft)	Distributed Leeward Force (lb/ft)
1	12	0.57	159.45	158.86
2	24	0.66	182.37	158.86
3	36	0.74	204.77	158.86
4	48	0.80	222.31	158.86
5	60	0.85	236.94	158.86
6	72	0.90	249.61	158.86
7	84	0.94	260.85	158.86
8	96	0.98	271.00	158.86
9	108	1.01	280.27	158.86
10	120	1.04	144.42	79.43
TOTAL			2,211.99	1,509.17

Table 6.6 Wind loads for y-direction winds (30-Storey Models)

Floor	z	K_z	Distributed Windward Force (lb/ft)	Distributed Leeward Force (lb/ft)
1	12	0.57	159.45	247.09
2	24	0.66	182.37	247.09
3	36	0.74	204.77	247.09
4	48	0.80	222.31	247.09
5	60	0.85	236.94	247.09
6	72	0.90	249.61	247.09
7	84	0.94	260.85	247.09
8	96	0.98	271.00	247.09
9	108	1.01	280.27	247.09
10	120	1.04	288.84	247.09
11	132	1.07	296.81	247.09
12	144	1.10	304.28	247.09
13	156	1.12	311.32	247.09
14	168	1.15	317.98	247.09
15	180	1.17	324.31	247.09
16	192	1.19	330.35	247.09
17	204	1.21	336.12	247.09
18	216	1.23	341.65	247.09
19	228	1.25	346.97	247.09
20	240	1.27	352.09	247.09
21	252	1.29	357.04	247.09
22	264	1.30	361.81	247.09
23	276	1.32	366.44	247.09
24	288	1.34	370.92	247.09
25	300	1.35	375.27	247.09
26	312	1.37	379.50	247.09
27	324	1.38	383.62	247.09
28	336	1.40	387.62	247.09
29	348	1.41	391.53	247.09
30	360	1.42	197.67	123.54
TOTAL			9,189.71	7,289.15

Table 6.7 Wind loads for y-direction winds (20-Storey Models)

Floor	z	K_z	Distributed Windward Force (lb/ft)	Distributed Leeward Force (lb/ft)
1	12	0.57	159.45	220.06
2	24	0.66	182.37	220.06
3	36	0.74	204.77	220.06
4	48	0.80	222.31	220.06
5	60	0.85	236.94	220.06
6	72	0.90	249.61	220.06
7	84	0.94	260.85	220.06
8	96	0.98	271.00	220.06
9	108	1.01	280.27	220.06
10	120	1.04	288.84	220.06
11	132	1.07	296.81	220.06
12	144	1.10	304.28	220.06
13	156	1.12	311.32	220.06
14	168	1.15	317.98	220.06
15	180	1.17	324.31	220.06
16	192	1.19	330.35	220.06
17	204	1.21	336.12	220.06
18	216	1.23	341.65	220.06
19	228	1.25	346.97	220.06
20	240	1.27	176.05	110.03
TOTAL			5,442.25	4,291.17

Table 6.8 Wind loads for y-direction winds (10-Storey Models)

Floor	z	K_z	Distributed Windward Force (lb/ft)	Distributed Leeward Force (lb/ft)
1	12	0.57	159.45	180.52
2	24	0.66	182.37	180.52
3	36	0.74	204.77	180.52
4	48	0.80	222.31	180.52
5	60	0.85	236.94	180.52
6	72	0.90	249.61	180.52
7	84	0.94	260.85	180.52
8	96	0.98	271.00	180.52
9	108	1.01	280.27	180.52
10	120	1.04	144.42	90.26
TOTAL			2,211.99	1,1714.94

K_z for the windward face is calculated at every floor as indicated before. Noting that the final floor's (roof) distributed force is divided by 2 since the force acting on beams come from both upper and below floor and the roof does not have a floor above.

Chapter 7

RESULTS AND THEIR INTERPRATATION

7.1 Results

During the study, the optimal configuration of lateral bracing systems in steel structures to resist wind forces is seeked. For this job three different structures (10, 20 and 30 stories) with twelve different bracing models are considered. The optimum designs of all three structures with all bracing models are made.

Due to stochastic nature of these techniques, each design example is independently solved five times and average performances are considered.

The tabulated results of the steel section for every group number of these three models can be seen Table 7.1 to Table 7.6. Optimum weights of the models are found by using OPTSTEEL and these weights are given in Table 7.7. The models having the minimum weight are model-G, model-E and model-D for 10, 20 and 30 stories respectively.

For each model cost calculations are carried out. This process is handled with formulas described in Chapter 5. As a result of the calculations the models leading to minimum costs are model-L for both 10 and 20 stories, model-D for the 30 floored structures. Costs of all models are presented in Table 7.8, Table 7.9 and Table 7.10.

Table 7.1 Member Grouping Details of 10-Storey Model-G

GROUP NUMBER	Member	Floor	GROUP NUMBER	Member	Floor
1	Inner Beams	1,2	19	XZ Outer Columns	7,8
2	Inner Beams	3,4	20	XZ Outer Columns	9,10
3	Inner Beams	5,6	21	YZ Outer Columns	1,2
4	Inner Beams	7,8	22	YZ Outer Columns	3,4
5	Inner Beams	9,10	23	YZ Outer Columns	5,6
6	Outer Beams	1,2	24	YZ Outer Columns	7,8
7	Outer Beams	3,4	25	YZ Outer Columns	9,10
8	Outer Beams	5,6	26	Inner Columns	1,2
9	Outer Beams	7,8	27	Inner Columns	3,4
10	Outer Beams	9,10	28	Inner Columns	5,6
11	Corner Columns	1,2	29	Inner Columns	7,8
12	Corner Columns	3,4	30	Inner Columns	9,10
13	Corner Columns	5,6	31	Bracings	1,2
14	Corner Columns	7,8	32	Bracings	3,4
15	Corner Columns	9,10	33	Bracings	5,6
16	XZ Outer Columns	1,2	34	Bracings	7,8
17	XZ Outer Columns	3,4	35	Bracings	9,10
18	XZ Outer Columns	5,6			

Table 7.2 Steel sections of 10-storey model-G

GROUP NUMBER	SECTION	CROSS-SECTIONAL AREA	GROUP NUMBER	SECTION	CROSS-SECTIONAL AREA
		(in^2)			(in^2)
1	W12X30	8.79	19	W18X40	11.8
2	W14X30	8.85	20	W8X21	6.16
3	W12X26	7.65	21	W14X99	29.1
4	W12X26	7.65	22	W24X84	24.7
5	W12X26	7.65	23	W14X68	20
6	W8X21	6.16	24	W10X49	14.4
7	W10X22	6.49	25	W12X45	13.1
8	W8X21	6.16	26	W21X166	48.8
9	W8X21	6.16	27	W24X104	30.6
10	W8X18	5.26	28	W12X72	21.1
11	W8X35	10.3	29	W21X50	14.7
12	W8X31	9.12	30	W8X31	9.12
13	W8X31	9.12	31	W8X24	7.08
14	W16X26	7.68	32	W6X15	4.45
15	W8X21	6.16	33	W8X31	9.12
16	W10X60	17.6	34	W8X31	9.12
17	W12X53	15.6	35	W8X31	9.12
18	W8X48	14.1			

Table 7.3 Member Grouping Details of 20-Storey Model-E

GROUP NUMBER	MEMBER	FLOOR	GROUP NUMBER	MEMBER	FLOOR
1	Inner Beams	1,2	36	XZ Outer Columns	11,12
2	Inner Beams	3,4	37	XZ Outer Columns	13,14
3	Inner Beams	5,6	38	XZ Outer Columns	15,16
4	Inner Beams	7,8	39	XZ Outer Columns	17,18
5	Inner Beams	9,10	40	XZ Outer Columns	19,20
6	Inner Beams	11,12	41	YZ Outer Columns	1,2
7	Inner Beams	13,14	42	YZ Outer Columns	3,4
8	Inner Beams	15,16	43	YZ Outer Columns	5,6
9	Inner Beams	17,18	44	YZ Outer Columns	7,8
10	Inner Beams	19,20	45	YZ Outer Columns	9,10
11	Outer Beams	1,2	46	YZ Outer Columns	11,12
12	Outer Beams	3,4	47	YZ Outer Columns	13,14
13	Outer Beams	5,6	48	YZ Outer Columns	15,16
14	Outer Beams	7,8	49	YZ Outer Columns	17,18
15	Outer Beams	9,10	50	YZ Outer Columns	19,20
16	Outer Beams	11,12	51	Inner Columns	1,2
17	Outer Beams	13,14	52	Inner Columns	3,4
18	Outer Beams	15,16	53	Inner Columns	5,6
19	Outer Beams	17,18	54	Inner Columns	7,8
20	Outer Beams	19,20	55	Inner Columns	9,10
21	Corner Columns	1,2	56	Inner Columns	11,12
22	Corner Columns	3,4	57	Inner Columns	13,14
23	Corner Columns	5,6	58	Inner Columns	15,16
24	Corner Columns	7,8	59	Inner Columns	17,18
25	Corner Columns	9,10	60	Inner Columns	19,20
26	Corner Columns	11,12	61	Bracings	1,2
27	Corner Columns	13,14	62	Bracings	3,4
28	Corner Columns	15,16	63	Bracings	5,6
29	Corner Columns	17,18	64	Bracings	7,8

Table 7.3 (Continued)

30	Corner Columns	19,20	65	Bracings	9,10
31	XZ Outer Columns	1,2	66	Bracings	11,12
32	XZ Outer Columns	3,4	67	Bracings	13,14
33	XZ Outer Columns	5,6	68	Bracings	15,16
34	XZ Outer Columns	7,8	69	Bracings	17,18
35	XZ Outer Columns	9,10	70	Bracings	19,20

Table 7.4 Steel Sections of 20-storey Model-E

GROUP NUMBER	SECTION	CROSS-SECTIONAL AREA	GROUP NUMBER	SECTION	CROSS-SECTIONAL AREA
		(in^2)			(in^2)
1	W18X35	10.3	36	W14X193	56.8
2	W12X30	8.79	37	W14X145	42.7
3	W18X46	13.5	38	W14X120	35.3
4	W12X35	10.3	39	W10X60	17.6
5	W14X43	12.6	40	W8X48	14.1
6	W24X68	20.1	41	W27X281	82.9
7	W24X55	16.3	42	W14X176	51.8
8	W21X50	14.7	43	W10X112	32.9
9	W14X30	8.85	44	W12X106	31.2
10	W18X35	10.3	45	W18X97	28.5
11	W8X18	5.26	46	W24X117	34.4
12	W8X21	6.16	47	W24X117	34.4
13	W8X21	6.16	48	W30X90	26.4
14	W18X35	10.3	49	W10X60	17.6
15	W24X68	20.1	50	W10X49	14.4
16	W24X62	18.3	51	W33X387	114
17	W10X22	6.49	52	W40X324	95.3
18	W21X44	13	53	W18X211	62.1
19	W8X21	6.16	54	W14X193	56.8
20	W12X26	7.65	55	W27X217	64
21	W30X124	36.5	56	W24X131	38.5
22	W27X102	30	57	W30X108	31.7
23	W12X120	35.3	58	W30X108	31.7
24	W21X101	29.8	59	W16X40	11.8
25	W14X145	42.7	60	W8X24	7.08
26	W27X146	43.1	61	W8X31	9.12
27	W12X65	19.1	62	W10X33	9.71
28	W12X65	19.1	63	W8X31	9.12

Table 7.4 (Continued)

29	W12X79	23.2	64	W8X31	9.12
30	W8X31	9.12	65	W8X28	8.24
31	W44X224	66.42	66	W6X20	5.89
32	W40X192	55.93	67	W8X21	6.16
33	W12X170	50	68	W6X20	5.89
34	W12X152	44.7	69	W6X15	4.45
35	W33X152	44.8	70	W4X13	3.83

Table 7.5 Member Grouping Details of 30-Storey Model-D

GROUP NUMBER	MEMBER	FLOOR	GROUP NUMBER	MEMBER	FLOOR
1	Inner Beams	1,2	54	XZ Outer Columns	17,18
2	Inner Beams	3,4	55	XZ Outer Columns	19,20
3	Inner Beams	5,6	56	XZ Outer Columns	21,22
4	Inner Beams	7,8	57	XZ Outer Columns	23,24
5	Inner Beams	9,10	58	XZ Outer Columns	25,26
6	Inner Beams	11,12	59	XZ Outer Columns	27,28
7	Inner Beams	13,14	60	XZ Outer Columns	29,30
8	Inner Beams	15,16	61	YZ Outer Columns	1,2
9	Inner Beams	17,18	62	YZ Outer Columns	3,4
10	Inner Beams	19,20	63	YZ Outer Columns	5,6
11	Inner Beams	21,22	64	YZ Outer Columns	7,8
12	Inner Beams	23,24	65	YZ Outer Columns	9,10
13	Inner Beams	25,26	66	YZ Outer Columns	11,12
14	Inner Beams	27,28	67	YZ Outer Columns	13,14
15	Inner Beams	29,30	68	YZ Outer Columns	15,16
16	Outer Beams	1,2	69	YZ Outer Columns	17,18
17	Outer Beams	3,4	70	YZ Outer Columns	19,20
18	Outer Beams	5,6	71	YZ Outer Columns	21,22
19	Outer Beams	7,8	72	YZ Outer Columns	23,24
20	Outer Beams	9,10	73	YZ Outer Columns	25,26
21	Outer Beams	11,12	74	YZ Outer Columns	27,28
22	Outer Beams	13,14	75	YZ Outer Columns	29,30
23	Outer Beams	15,16	76	Inner Columns	1,2
24	Outer Beams	17,18	77	Inner Columns	3,4
25	Outer Beams	19,20	78	Inner Columns	5,6
26	Outer Beams	21,22	79	Inner Columns	7,8
27	Outer Beams	23,24	80	Inner Columns	9,10
28	Outer Beams	25,26	81	Inner Columns	11,12
29	Outer Beams	27,28	82	Inner Columns	13,14

Table 7.5 (Continued)

30	Outer Beams	29,30	83	Inner Columns	15,16
31	Corner Columns	1,2	84	Inner Columns	17,18
32	Corner Columns	3,4	85	Inner Columns	19,20
33	Corner Columns	5,6	86	Inner Columns	21,22
34	Corner Columns	7,8	87	Inner Columns	23,24
35	Corner Columns	9,10	88	Inner Columns	25,26
36	Corner Columns	11,12	89	Inner Columns	27,28
37	Corner Columns	13,14	90	Inner Columns	29,30
38	Corner Columns	15,16	91	Bracings	1,2
39	Corner Columns	17,18	92	Bracings	3,4
40	Corner Columns	19,20	93	Bracings	5,6
41	Corner Columns	21,22	94	Bracings	7,8
42	Corner Columns	23,24	95	Bracings	9,10
43	Corner Columns	25,26	96	Bracings	11,12
44	Corner Columns	27,28	97	Bracings	13,14
45	Corner Columns	29,30	98	Bracings	15,16
46	XZ Outer Columns	1,2	99	Bracings	17,18
47	XZ Outer Columns	3,4	100	Bracings	19,20
48	XZ Outer Columns	5,6	101	Bracings	21,22
49	XZ Outer Columns	7,8	102	Bracings	23,24
50	XZ Outer Columns	9,10	103	Bracings	25,26
51	XZ Outer Columns	11,12	104	Bracings	27,28
52	XZ Outer Columns	13,14	105	Bracings	29,30
53	XZ Outer Columns	15,16			

Table 7.6 Steel Sections of 30-floored Model-D

GROUP NUMBER	SECTION	CROSS-SECTIONAL AREA	GROUP NUMBER	SECTION	CROSS-SECTIONAL AREA
		(in^2)			(in^2)
1	W24x62	18.3	54	W14x257	75.6
2	W18x35	10.3	55	W36x232	68.1
3	W14x38	11.2	56	W14x145	42.7
4	W14x43	12.6	57	W14x233	68.5
5	W21x62	18.3	58	W12x96	28.2
6	W40x249	73.3	59	W14x132	38.8
7	W16x36	10.6	60	W14x90	26.5
8	W30x108	31.7	61	W36x393	116
9	W21x50	14.7	62	W40x277	81.4
10	W24x55	16.3	63	W36x182	53.6
11	W16x40	11.8	64	W24x146	43
12	W30x116	34.2	65	W36x182	53.6
13	W18x35	10.3	66	W27x217	64
14	W33x118	34.7	67	W27x178	52.5
15	W14x30	8.85	68	W40x215	63.4
16	W10x22	6.49	69	W40x328	96.79
17	W21x44	13	70	W40x215	63.4
18	W12x26	7.65	71	W44x285	85.12
19	W30x90	26.4	72	W27x102	30
20	W33x118	34.7	73	W40x149	43.8
21	W30x90	26.4	74	W30x173	51
22	W36x135	39.7	75	W14x38	11.2
23	W21x62	18.3	76	W40x593	174
24	W14x34	10	77	W33x515	151
25	W44x198	58.38	78	W21x248	72.8
26	W24x62	18.3	79	W18x158	46.3
27	W24x62	18.3	80	W30x235	69.2

Table 7.6 (Continued)

28	W21x44	13	81	W40x244	71.47
29	W30x90	26.4	82	W36x182	53.6
30	W8x18	5.26	83	W36x135	39.7
31	W40x328	96.79	84	W44x285	85.12
32	W40x362	107	85	W24x131	38.5
33	W40x480	140	86	W27x102	30
34	W33x468	137	87	W24x117	34.4
35	W30x581	170	88	W10x39	11.5
36	W14x398	117	89	W33x130	38.3
37	W14x370	109	90	W14x34	10
38	W33x354	104	91	W12x45	13.1
39	W12x152	44.7	92	W10x45	13.3
40	W24x131	38.5	93	W10x49	14.4
41	W14x120	35.3	94	W8x48	14.1
42	W14x159	46.7	95	W8x31	9.12
43	W14x109	32	96	W12x26	7.65
44	W14x99	29.1	97	W8x31	9.12
45	W8x35	10.3	98	W14x68	20
46	W33x354	104	99	W10x39	11.5
47	W14x370	109	100	W8x31	9.12
48	W40x328	96.79	101	W12x26	7.65
49	W24x335	98.4	102	W10x39	11.5
50	W40x298	87.8	103	W12x40	11.7
51	W36x720	211	104	W6x15	4.45
52	W27x336	98.9	105	W6x15	4.45
53	W36x300	88.3			

Table 7.7 Optimum Weights of the Models

Model	Weight(lbs)	Model	Weight(lbs)	Model	Weight(lbs)
A10	324,359.70	A20	1,265,589.63	A30	3,400,000.00
B10	272,702.62	B20	998,232.87	B30	2,776,622.69
C10	286,375.54	C20	1,033,941.75	C30	2,823,142.45
D10	284,646.58	D20	997,485.34	D30	2,671,861.58
E10	264,378.75	E20	968,338.72	E30	2,723,415.28
F10	294,735.53	F20	1,082,734.07	F30	3,044,796.17
G10	256,364.87	G20	988,401.56	G30	2,784,166.67
H10	331,129.62	H20	1,512,742.91	H30	3,864,288.44
I10	328,862.27	I20	1,403,282.56	I30	3,705,856.27
J10	333,309.78	J20	1,550,934.86	J30	4,136,391.64
K10	291,845.03	K20	1,130,975.15	K30	3,360,367.92
L10	272,399.64	L20	1,078,923.83	L30	3,012,487.42
M10	305,280.04	M20	1,305,087.52	M30	3,608,887.99

Table 7.8 Cost of 10-Storey Models

Model	Element Mass (kg)	Element Cost (Euro)	Joint Cost (Euro)	Transportation Cost (Euro)	Erection Cost (Euro)	Extra Costs (Euro)	Total Cost(Euro)
A-10	147,127.09	58,850.83	19,002.57	3,569.09	27,300.31	29,850.58	**138,573.39**
B-10	123,695.83	49,478.33	19,002.57	3,053.60	23,357.29	25,539.23	**120,431.03**
C-10	129,897.76	51,959.10	19,002.57	3,190.05	24,400.96	26,680.39	**125,233.06**
D-10	129,113.52	51,645.41	19,002.57	3,172.79	24,268.98	26,536.09	**124,625.84**
E-10	119,920.18	47,968.07	19,002.57	2,970.54	22,721.93	24,844.51	**117,507.63**
F-10	133,689.79	53,475.91	19,002.57	3,273.47	25,039.08	27,378.12	**128,169.15**
G-10	116,285.15	46,514.06	19,002.57	2,890.57	22,110.23	24,175.67	**114,693.09**
H-10	150,197.87	60,079.15	2,529.66	3,322.66	25,415.31	27,789.50	**119,136.26**
I-10	149,169.42	59,667.77	2,529.66	3,300.03	25,242.24	27,600.26	**118,339.95**
J-10	151,186.77	60,474.71	2,529.66	3,344.41	25,581.72	27,971.45	**119,901.95**
K-10	132,378.68	52,951.47	12,406.12	3,124.34	23,898.35	26,130.84	**118,511.12**
L-10	123,558.40	49,423.36	12,406.12	2,930.29	22,414.08	24,507.90	**111,681.75**
M-10	138,472.70	55,389.08	12,406.12	3,258.41	24,923.85	27,252.13	**123,229.60**

Table 7.9 Cost of 20-Storey Models

	Element Mass (kg)	Element Cost (Euro)	Joint Cost (Euro)	Transportation Cost (Euro)	Erection Cost (Euro)	Extra Costs (Euro)	Total Cost(Euro)
A-20	574,061.80	229,624.72	97,875.32	14,268.19	109,138.71	119,333.98	**570,240.93**
B-20	452,790.81	181,116.32	97,875.32	11,600.23	88,731.23	97,020.12	**476,343.22**
C-20	468,988.09	187,595.24	97,875.32	11,956.57	91,456.90	100,000.42	**488,884.45**
D-20	452,451.74	180,980.70	97,875.32	11,592.77	88,674.17	96,957.73	**476,080.69**
E-20	439,231.06	175,692.42	97,875.32	11,301.92	86,449.39	94,525.13	**465,844.18**
F-20	491,119.91	196,447.96	97,875.32	12,443.47	95,181.25	104,072.67	**506,020.68**
G-20	448,331.41	179,332.56	97,875.32	11,502.12	87,980.80	96,199.59	**472,890.40**
H-20	686,168.64	274,467.46	7,741.61	15,132.32	115,748.48	126,561.21	**539,651.06**
I-20	636,518.26	254,607.31	7,741.61	14,040.01	107,393.31	117,425.54	**501,207.77**
J-20	703,492.22	281,396.89	7,741.61	15,513.44	118,663.69	129,748.74	**553,064.37**
K-20	513,001.70	205,200.68	65,781.87	12,379.46	94,691.58	103,537.26	**481,590.84**
L-20	489,391.62	195,756.65	65,781.87	11,860.03	90,718.47	99,193.01	**463,310.02**
M-20	591,977.74	236,791.10	65,781.87	14,116.93	107,981.67	118,068.85	**542,740.41**

Table 7.10 Cost of 30-Storey Models

	Element Weight (kg)	Element Cost (Euro)	Joint Cost (Euro)	Transportation Cost (Euro)	Erection Cost (Euro)	Extra Costs (Euro)	Total Cost(Euro)
A-30	1,542,214.06	616,885.62	238,836.71	37,348.40	285,681.31	312,368.44	**1,491,120.48**
B-30	1,259,454.86	503,781.95	238,836.71	31,127.70	238,098.59	260,340.75	**1,272,185.70**
C-30	1,280,555.88	512,222.35	238,836.71	31,591.92	241,649.47	264,223.34	**1,288,523.79**
D-30	1,211,936.02	484,774.41	238,836.71	30,082.28	230,102.12	251,597.28	**1,235,392.81**
E-30	1,235,320.39	494,128.16	238,836.71	30,596.74	234,037.24	255,900.01	**1,253,498.85**
F-30	1,381,096.31	552,438.52	238,836.71	33,803.81	258,568.42	282,722.78	**1,366,370.23**
G-30	1,262,876.76	505,150.70	238,836.71	31,202.98	238,674.43	260,970.38	**1,274,835.20**
H-30	1,752,811.75	701,124.70	33,343.35	38,616.77	295,383.19	322,976.63	**1,391,444.63**
I-30	1,680,948.13	672,379.25	33,343.35	37,035.77	283,289.98	309,753.72	**1,335,802.07**
J-30	1,876,235.69	750,494.28	33,343.35	41,332.10	316,152.97	345,686.63	**1,487,009.32**
K-30	1,524,237.25	609,694.90	157,539.39	35,647.12	272,668.09	298,139.58	**1,373,689.08**
L-30	1,366,441.31	546,576.52	157,539.39	32,175.61	246,114.19	269,105.13	**1,251,510.84**
M-30	1,636,964.06	654,785.62	157,539.39	38,127.11	291,637.76	318,881.31	**1,460,971.19**

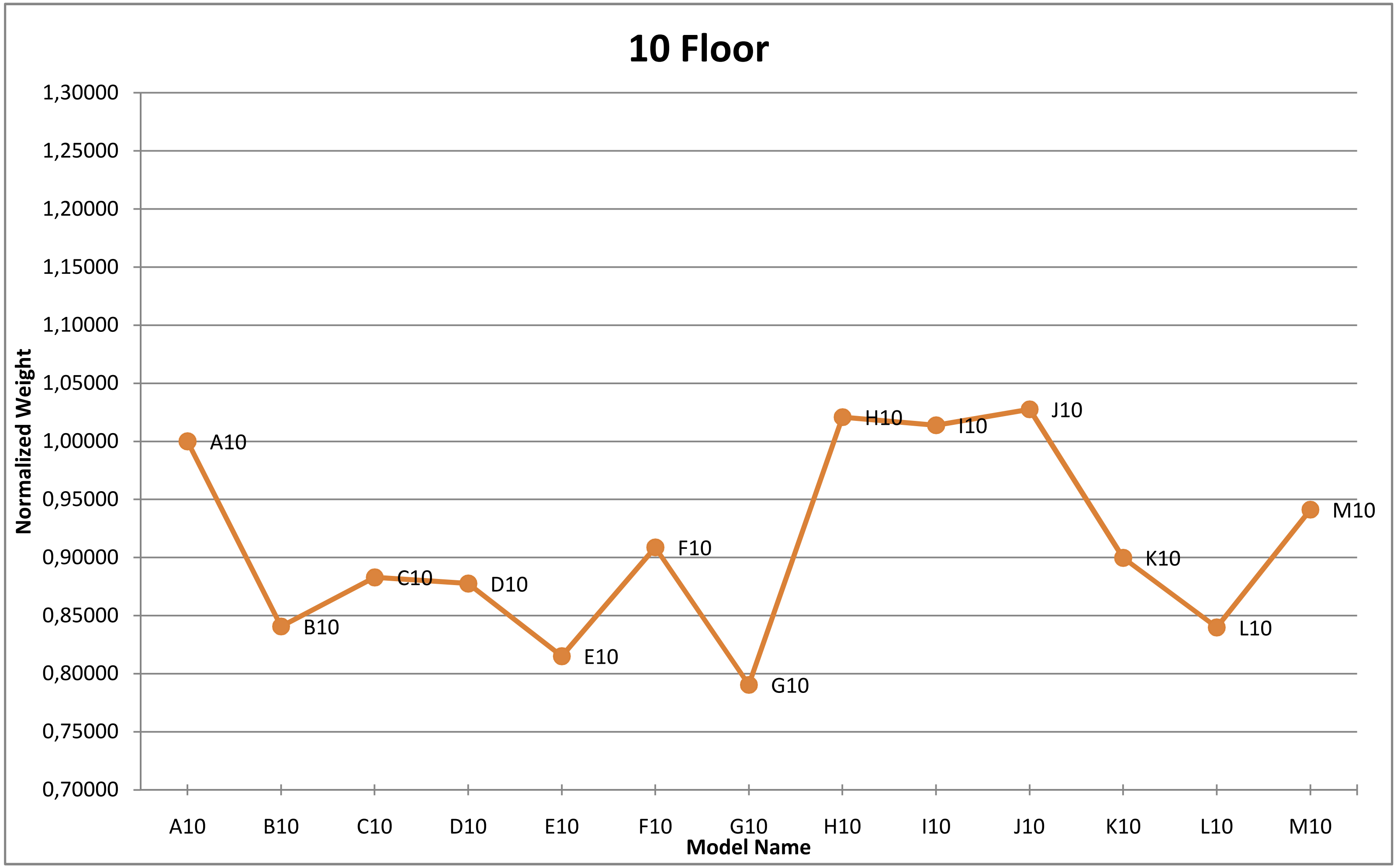


Figure 7.1 Normalized Weight vs. Model (10- Storey)

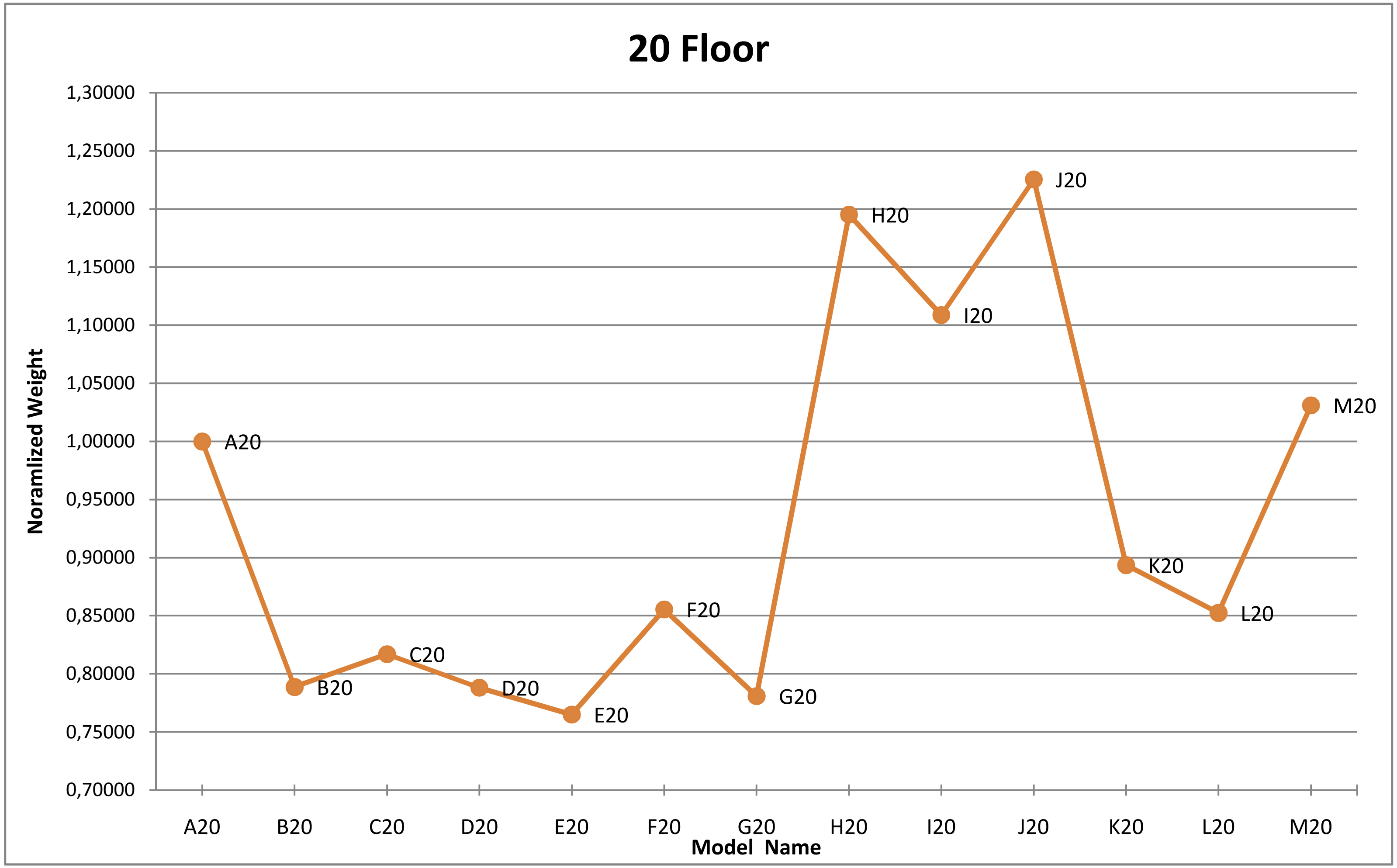


Figure 7.2 Normalized Weight vs. Model (20- Storey)

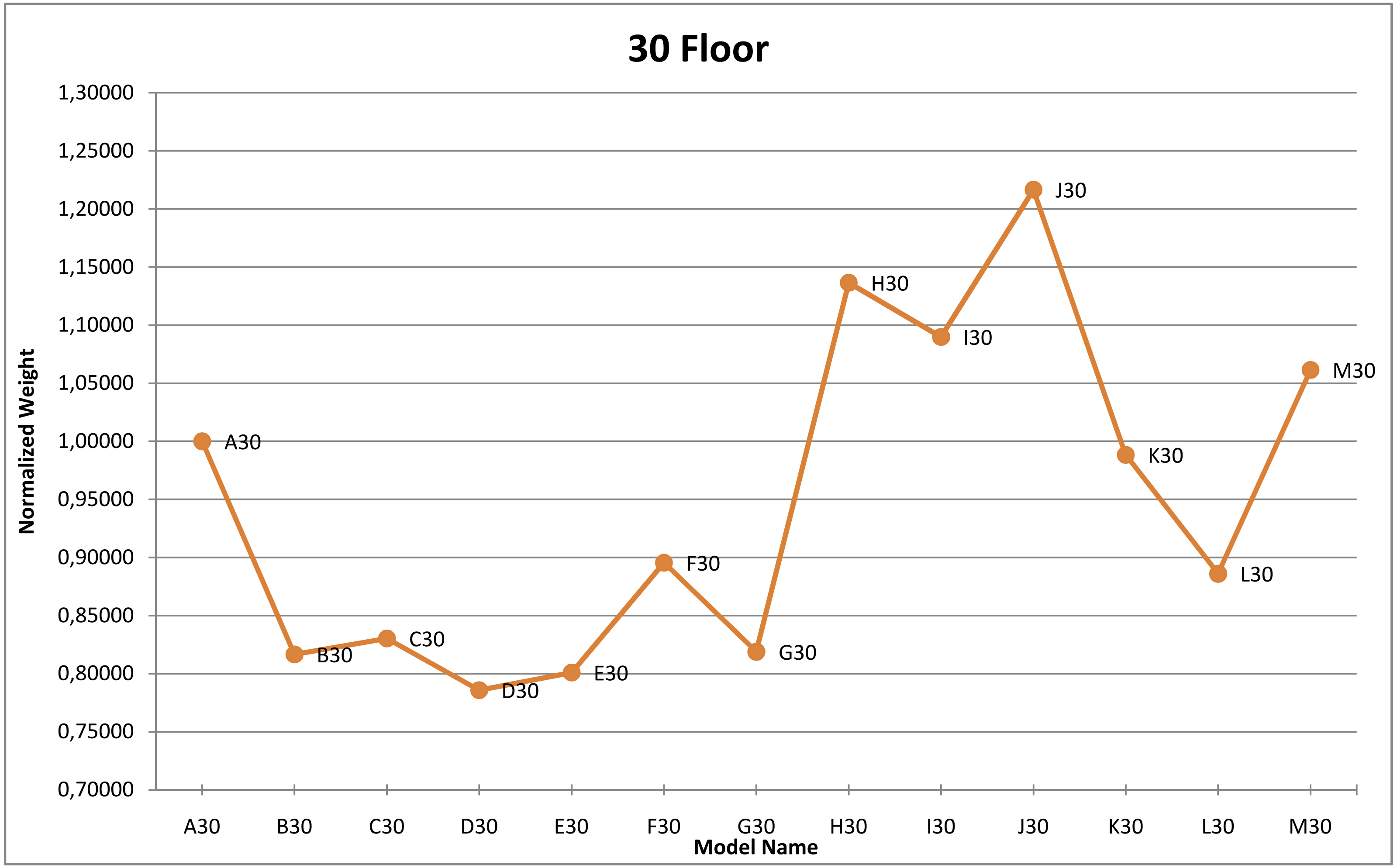


Figure 7.3 Normalized Weight vs. Model (30- Storey)

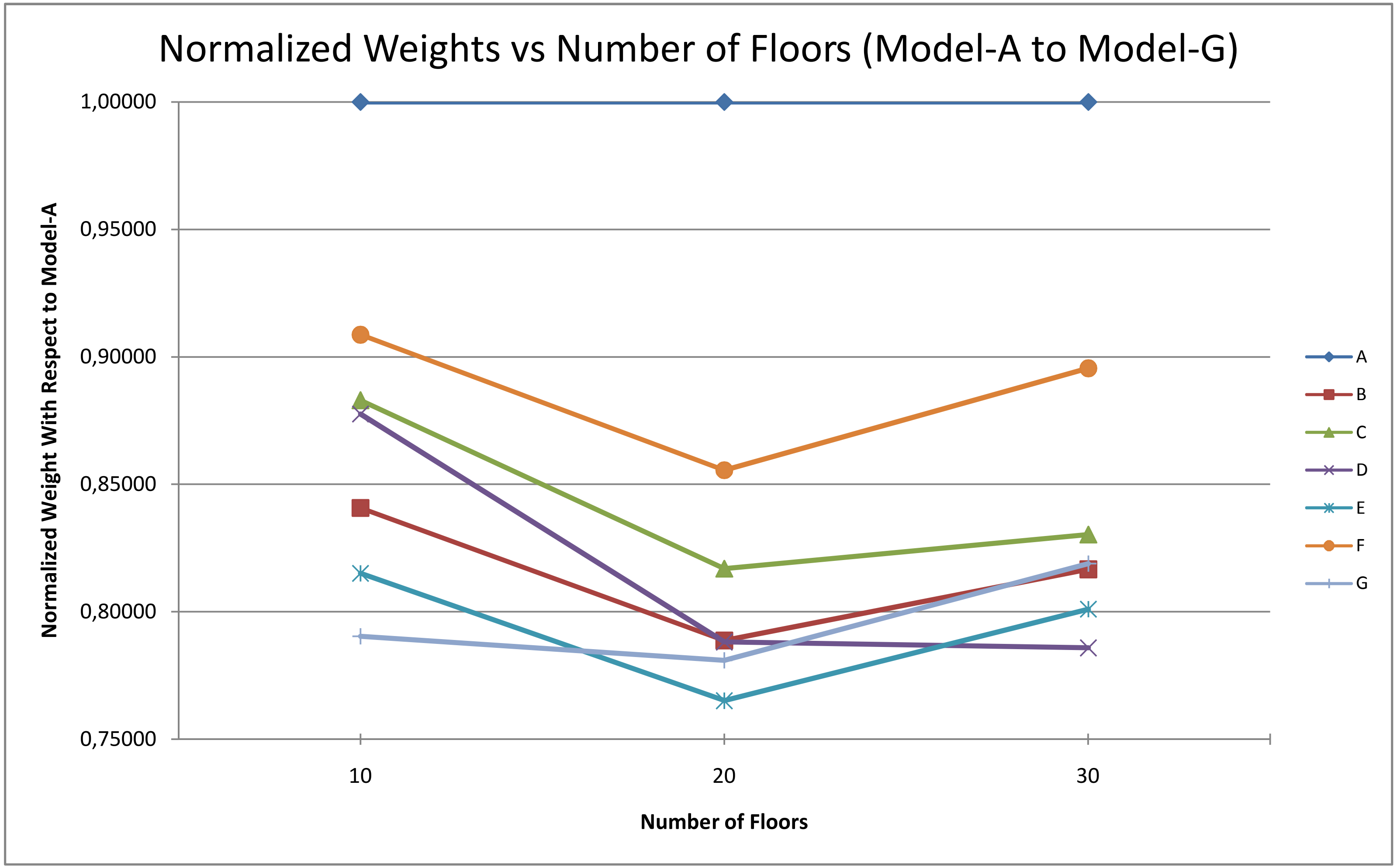


Figure 7.4 Normalized Weight vs. Number of Floors (Model-A to Model-G)

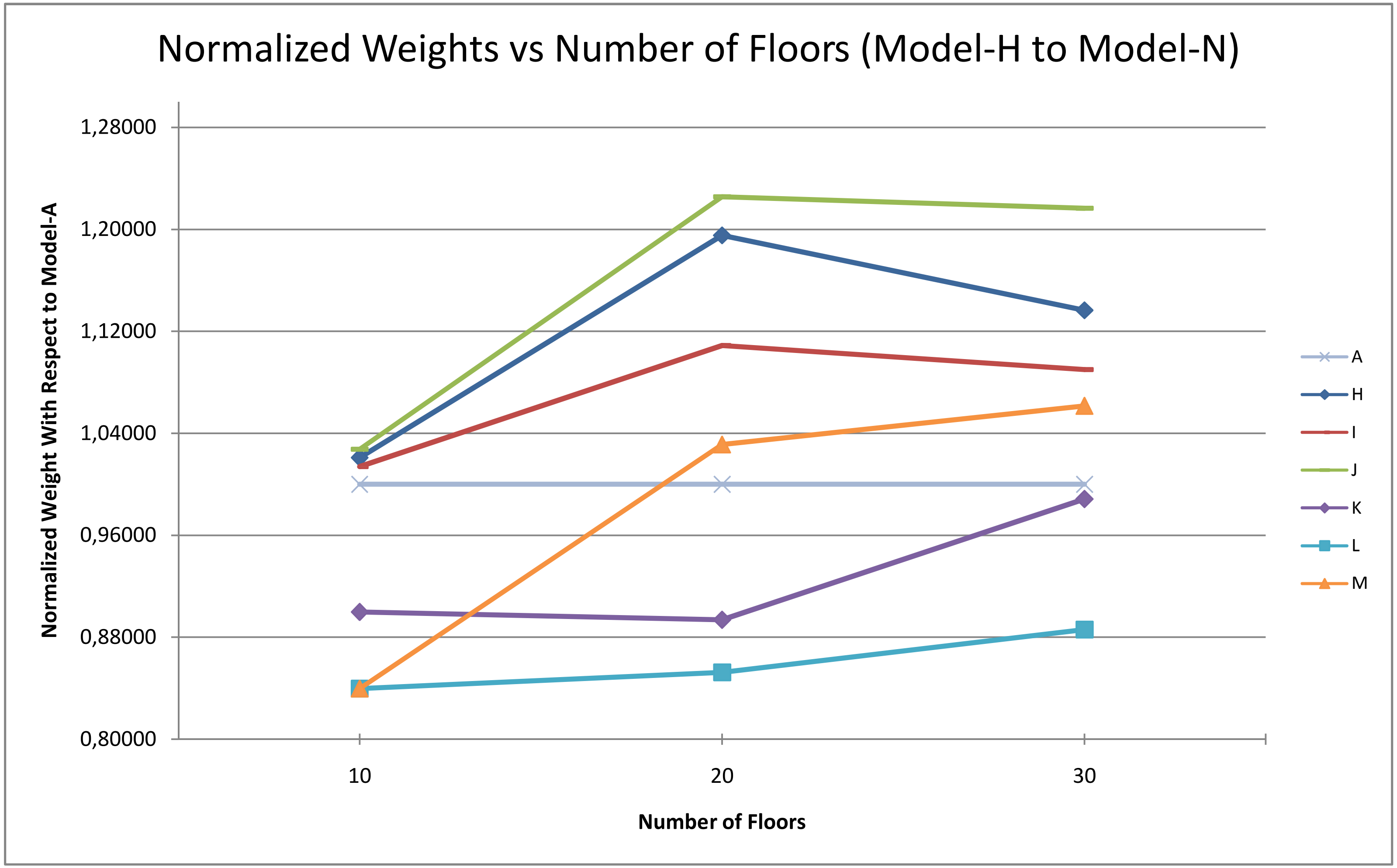


Figure 7.5 Normalized Weight vs. Number of Floors (Model-H to Model-N)

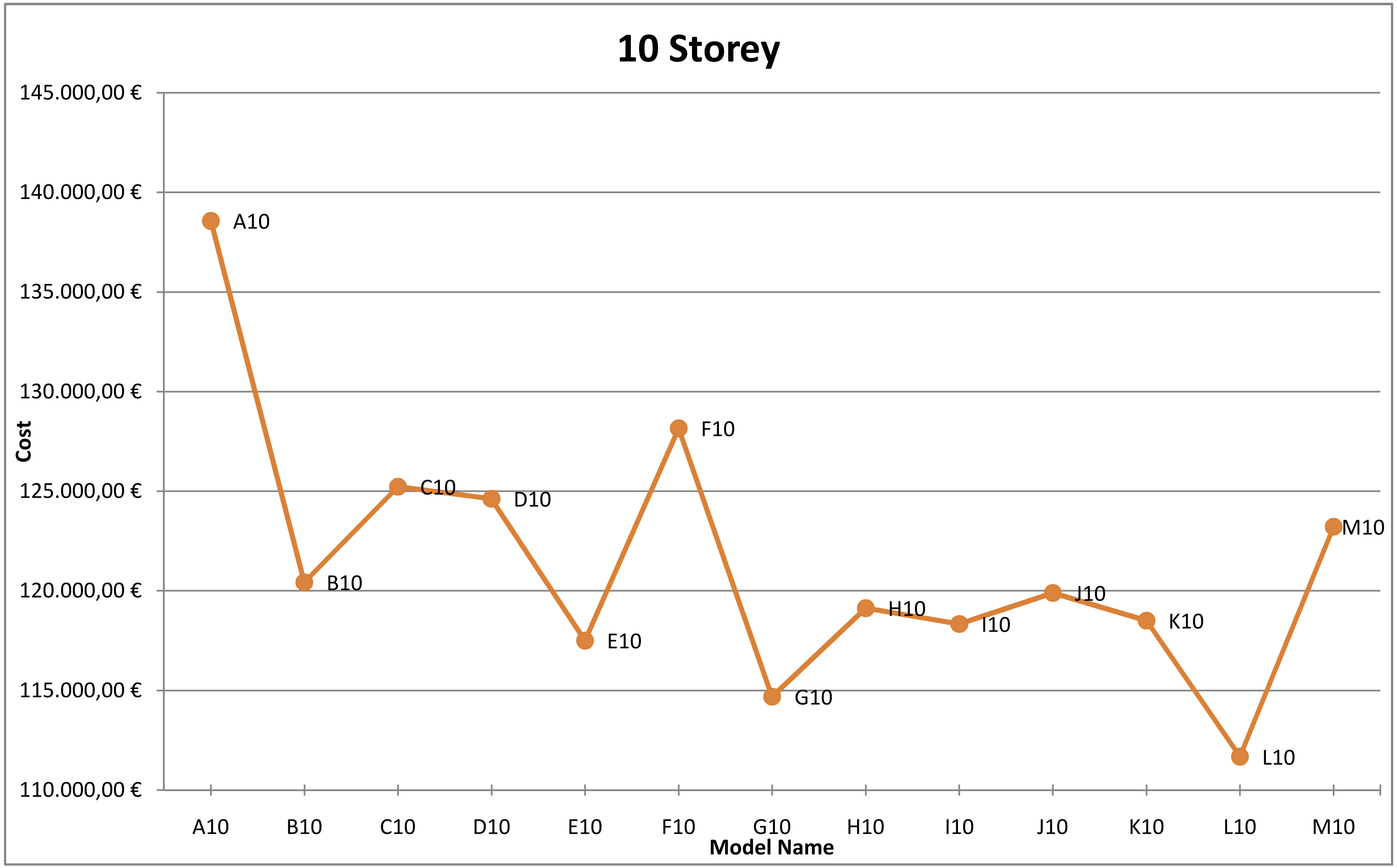


Figure 7.6 Cost of the Models (10-Storey)

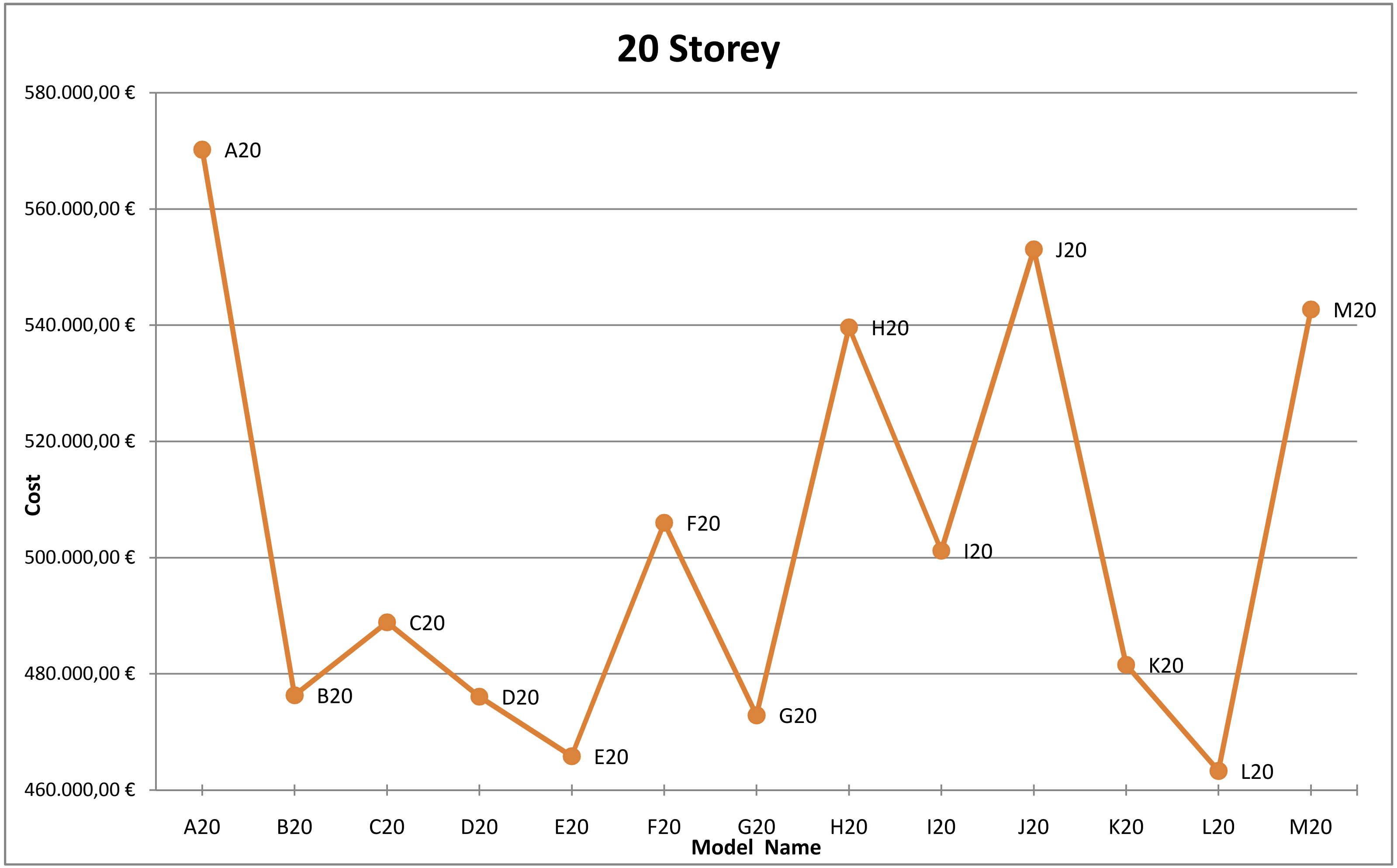


Figure 7.7 Cost of the Models (20-Storey)

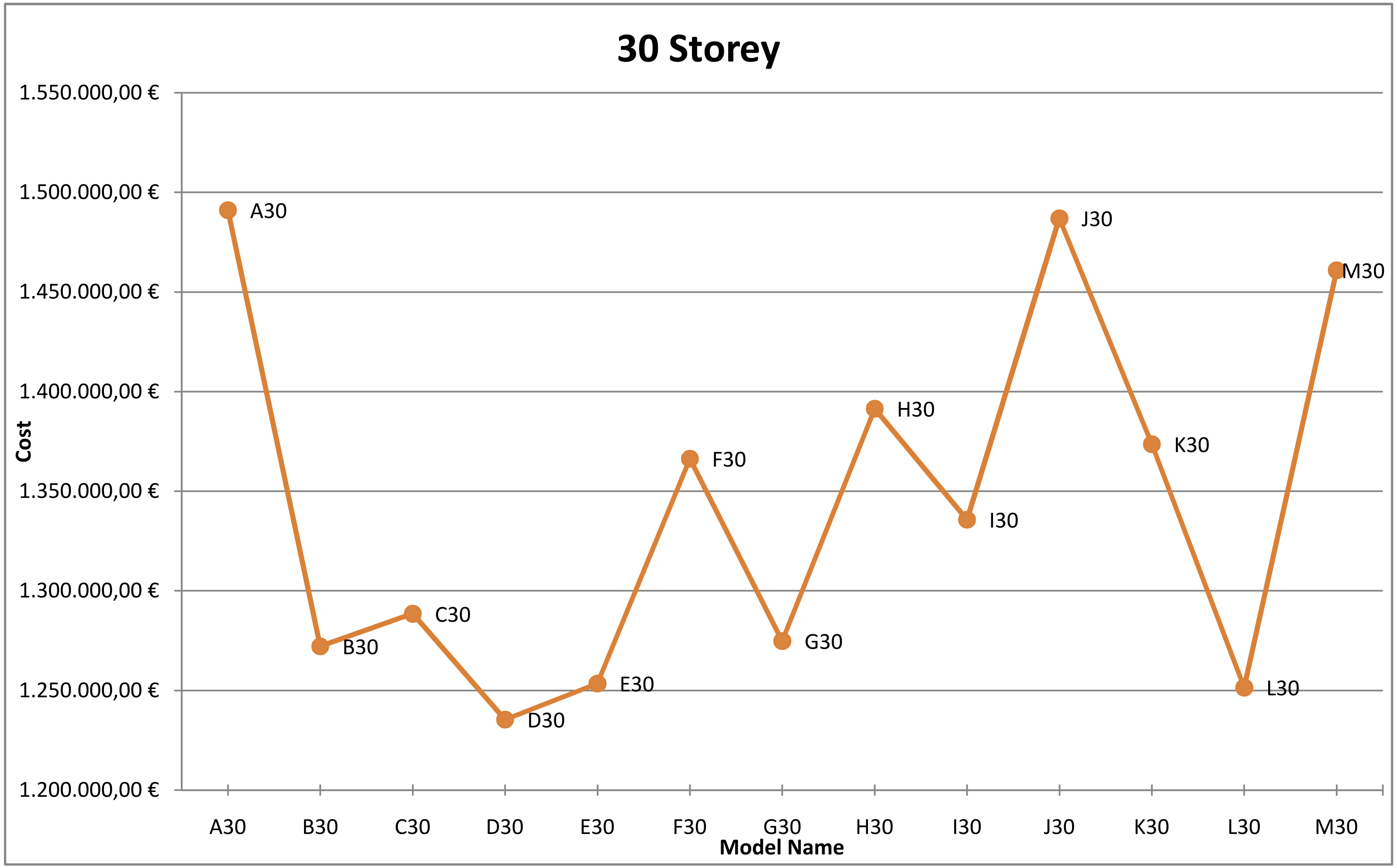


Figure 7.8 Cost of the Models (30-Storey)

7.2 Interpretation of Results

The results of the weight optimization problems and cost analysis of models are interpreted from Figure 7.1 to Figure 7.8. From Figure 7.1 to Figure 7.3, the normalized weights of the models are shown. The normalized weight is the weight of the specific model divided by the weight of the model-A having the same number of floors.

Figure 7.4 and Figure 7.5 shows the change of the normalized weight of the model with the structure height. With this figures, the estimation of the performance increase or decrease of the models with the structure height is easier.

The values of the cost analysis of the structures are shown in Figure 7.6, Figure 7.7 and Figure 7.8.

Below conclusions can be obtained from the outputs (The structures are named according to the bracing models they have):

- For 10-Floor Structures (Figure 7.1 and Figure 7.6):
  - The model having least weight is the model-G and the heaviest frame is model-J.
  - The best bracing model is model-L (knee-type). Although it does not have the least weight, it is the cheapest model among 10-floored ones.
  - For low-rise structures there is no need to use rigid beam-column connections. Those models yield to expensive frames.
  - The importance of the bracing model choice is not very significant in 10-floored models. The fluctuation in the cost are small compared to the 20 and 30-floored models.
  - From model-B to model-G the weights are less than the weight of the model-A (i.e. without bracing). This is an expected results since these models are rigid jointed, i.e. same as model-A, moreover they have bracings.

- For the pin jointed models, model-K, model-L and model-M are lighter than the model-A. The rest of the models are heavier than the model-A.
- For low-rise structures, there is no need for the outrigger trusses. In Figure 7.1, it can be seen that the model-B gives better result than the model-C and model-D in case of weight. Outrigger truss gives extra weight to the structure.

• For 20-Floor Structures (Figure 7.2 and Figure 7.7):
    - Model having the least weight is model-E and the heaviest model is model-J.
    - The model-L leads to cheapest frame among 20-floored models, although it ranks sixth in case of the structural weight.
    - As the structure height increases, the cost of the models having rigid beam-column connections decrease relative to the other models.
    - From model-B to model-G the weights are less than the weight of the model-A (i.e. without bracing). This is an expected results since these models are rigid jointed, i.e. same as model-A, moreover they have bracings.
    - For the pin jointed models, only model-K and model-L are lighter than the model-A. The rest of the models are heavier than the model-A.

• For 30-Floor structure (Figure 7.3 and Figure 7.8):
    - Model-F has the least weight and the worst one in case of weight is model-J.
    - From model-B to model-G the weights are less than the weight of the model-A (i.e. without bracing). This is an expected result as explained before.
    - The rest of the models (model-H to model-M) are heavier than the model-A. This indicates that the pin jointed structures have less lateral stability than the rigid jointed ones as expected.

- The cheapest frame of 30-floored models is model-D. This shows that the importance of rigid beam-column connections increases with the structure height.
- As can be seen from the Figure 7.3, the model-B and model-C has similar weights but model-D has less weight then both. The reason for this is that when outrigger truss is introduced, the structure is expected to have more lateral stability. But the model-D having less weight than the model-C can be explained as that the optimum placing of outrigger is in the middle rather than the top floor. This is also indicated in (Taranath, Steel, concrete, and composite design of tall buildings, 1997) in section 4.8.4.

- General Conclusions:
  - All of the rigid jointed frames with bracings have less weight than the rigid jointed model without bracing (model-A). That is an expected result.
  - Model-F does not give good results. Among the rigid jointed models, it is the worst model. It is known that knee-type braces does not have high performance. They are rather used in case of serviceability constraints.
  - The performance of model-D (outrigger at mid-height) increases significantly with the structure height. It is the most economical one among the 30-floored models.
  - Model-G gives very good result for low-rise structure. With increasing height, it yields to worse results. That means Z-type bracings are more applicable to low-rise buildings. It is second economic model among 10-storey ones.
  - Model-E is always better than the model-B. This can be explained as that when the middle span is supported with K-type framework, the effective length of the middle beam is decreased. This yields to a more rigid element.

- It is obvious that the structure having minimum weight does not imply the most economical structure. Among 20-storey models model-L is heavier than almost all of the rigid jointed models but it is the most economical one.
- Structural weight is considered to be the indication of the structure cost. This is not %100 true, since the study reveals that, models having more weight may lead to lower cost.
- As stated before there are 4 different types of structural systems in this study. These are moment-resisting beam-column connections (Type-1) (model-A), braced frames with moment-resisting connections (Type-2) (model-B to model-G), braced frames with pin-jointed connections (Type-3) (model-H to model-J) and braced frames with both pin-jointed and moment-resisting connections (Type-4) (model-K to model-M). From each type, three different bracing models are chosen such that:
    - From Type-2: model-B, model-E and model-F
    - From Type-3: model-H, model-I and model-J
    - From Type4: model-K, model-L and model-L

  Average weights of them are calculated. It is obvious that as the structure height increases, the performance of the models that have rigid beam-column connections increase. The values are given in Table 7.11.

Table 7.11Average Normalized Weights and Costs of Sample Three Models from Each Bracing Types

Model	Average Normalized Weight of Three Models	Average Cost of Three Models
B10	0.85482	122,035.94
E10		
F10		
H10	1.02078	119,126.05
I10		
J10		
K10	0.89358	117,807.49
L10		
M10		
B20	0.80313	482,736.02
E20		
F20		
H20	1.17651	531,307.73
I20		
J20		
K20	0.92578	495,880.42
L20		
M20		
B30	0.83772	1,297,351.59
E30		
F30		
H30	1.14769	1,404,752.00
I30		
J30		
K30	0.97860	1,362,057.03
L30		
M30		

Chapter 8

CONCLUSIONS AND SUMMARY

Since the design of structures is a challenging task, not all the designers can manage to make a perfect or optimum design. This yields to non-economical structures since the sections that are used are larger than they supposed to be. Since economy is one of the most important considerations of the design process, designers should find a way of making more economical designs. This is possible only if he/she uses some optimization techniques.

The main starting point of the ESs is to solve problems that have real-valued parameter. After some time, ESs were developed to solve problems that has discrete parameters. For a structural design problem, there is definite number of members and there is generally a steel set of sections that the designer chooses from. This is a typical example of a problem having discrete parameters.

The aim of this study is to determine the optimal configuration of lateral bracing models in steel structures subjected to wind forces. For this purpose three different structures (10, 20 and 30 floored) are prepared and optimum designs of them with twelve different bracing models are carried out. The wind speed is assumed to be 125 mph and as a lateral load, only wind load is taken into consideration. In case of having other types of loads like earthquake loads, the performances of the models might change slightly. For all design examples, the following material properties of the steel are used: modulus of elasticity (E) = 29,000 ksi (203,893.6 MPa) and yield stress (F_y) = 36 ksi (253.1 MPa). The detailed results and their discussions are given in Chapter 7.

General conclusions of the results are as below:

- Bracing always decreases the weight
- For low-rise structures, there is no need to use rigid connections
- For low-rise structures, the bracing choice is not very important
- There is no need for an outrigger truss in low-rise models
- Z-type bracings are more applicable to low-rise structures
- Model having a high weight may lead to a low-cost model. Typical example to this case is L-20
- Pin-jointed structures are always heavier than the rigid jointed ones
- Importance of the rigid connections increases with the structure height
- Knee-type bracing does not give good results. We only consider of using this type of bracing when there is space limitation

Summary of the thesis is given below:

In Chapter 2, the optimum design of steel frames is explained. First, optimization is explained. Details of it is given. Then the design optimization of steel frames is given and problem formulation is indicated. The constraints of the problem are given in this chapter also. Evolutionary Algorithms is explained and then detailed description of the ESs is given. The step-by-step procedure of ESs is introduced. Then the operators of it such as recombination and mutation are explained. The differences between continuous and discrete approaches are outlined in this chapter also.

In Chapter 3, design loads that are used on the test structures are outlined. There are mainly four load types that are used in our structures. As gravity loads; dead load, live load and snow load are applied. On the other hand, as the horizontal force the wind loads are chosen. In this chapter, calculations of these loads are given in detail. First the wind loads are explained. The importance of wind loads on structures is given. After that the step-by-step procedure of the wind load calculation is outlined according to ASCE 7 method 2. The wind loading values of the structures are presented in tables. After that the dead, live and snow load calculations are given and the values of them are tabulated.

In Chapter 4, the lateral force resisting systems are outlined. The reason for the usage of bracing systems in structures is given. After that the bracing models in the literature are introduced. There are mainly four types of systems that are used in our study: (i) moment-resisting beam-column connections, (ii) braced frames with moment-resisting connections, (iii) braced frames with pin-jointed connections and (iv) braced frames with both pin-jointed and moment-resisting connections. These systems with different brace arrangements like x-type, z-type, k-type are studied in this study.

In Chapter 5 cost analysis of steel frames is described. In many cases, structure with best design weight is pretended to be the most economic one. However, this is only an assumption that the minimal weight could lead to reasonable low overall costs. Other costs such as joint costs can differ from structure to structure such that it can affect the total cost significantly. Here, in this chapter cost of a structure is divided in to five parts. These are total material cost of the elements, total material and manufacturing cost of joints, total transportation costs, total erection costs and extra costs. Cost of the structure can be defined as the sum of all these four elements.

In Chapter 6 the details of the structural models are given. Three different structures with 10, 20 and 30 stories are considered and for every structure all of the prescribed bracing models are applied. Members are grouped such that inner beams, outer beams, corner columns, x-z outer columns, y-z outer columns, inner columns and bracing members at every two floors have the same cross-section. There are two load cases. In the first load case, the gravity loads are applied with the wind loads acting along x-axis, in the second one they are applied with the wind forces acting along y-axis.

As stated before the results of the analyses and discussions are given in Chapter 7.

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APPENDIX A

OVERVIEW OF THE SOFTWARE, OPTSTEEL

The optimization software is called OPTSTEEL, Optimum Design of Steel Structural Systems in Parallel Computing Environment, designed by Dr. O. Hasançebi from Middle East Technical University.

OPTSTEEL is software capable of making the optimal design of a structure using Evolution Strategies, which is one of the techniques of Evolutionary Algorithms. The main screen of the software can be seen in Figure A-1:

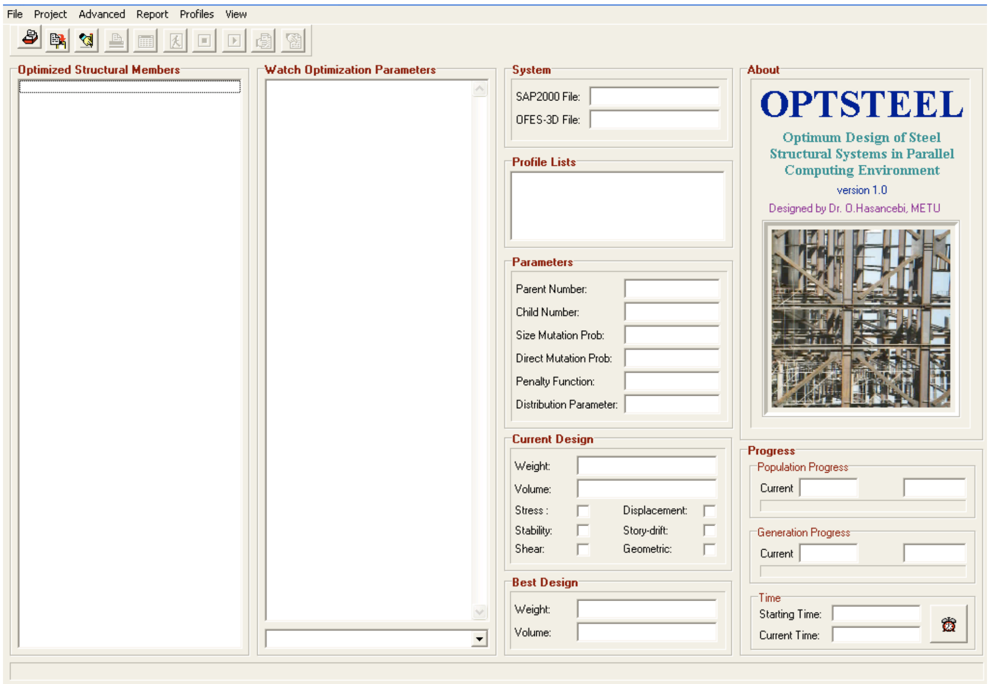

Figure A-1 Main Screen of OPTSTEEL

After creating the model in SAP2000 and assigning the loads, the .s2k file is recalled from the OPTSTEEL. Then the constraints are defined in OPTSTEEL such that:

- Sidesway along both x and y-axis is prevented for braced frames.
- Stress, stability and shear checks are made according to the selected code, which is AISC-ASD.
- Geometric check (beam-column connection) is made.
- Story drift check is made for both directions and the relative story drift value is 0.0025.
- Maximum displacement checks are made for both directions. This value is defined as the (building height)/400.

For the computational time advantages, the structural analyses are made in SAP2000. 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. Therefore the models of the structures are created in SAP2000. Some screen shots of SAP-models can be seen in Figure A-2 and Figure A-3.

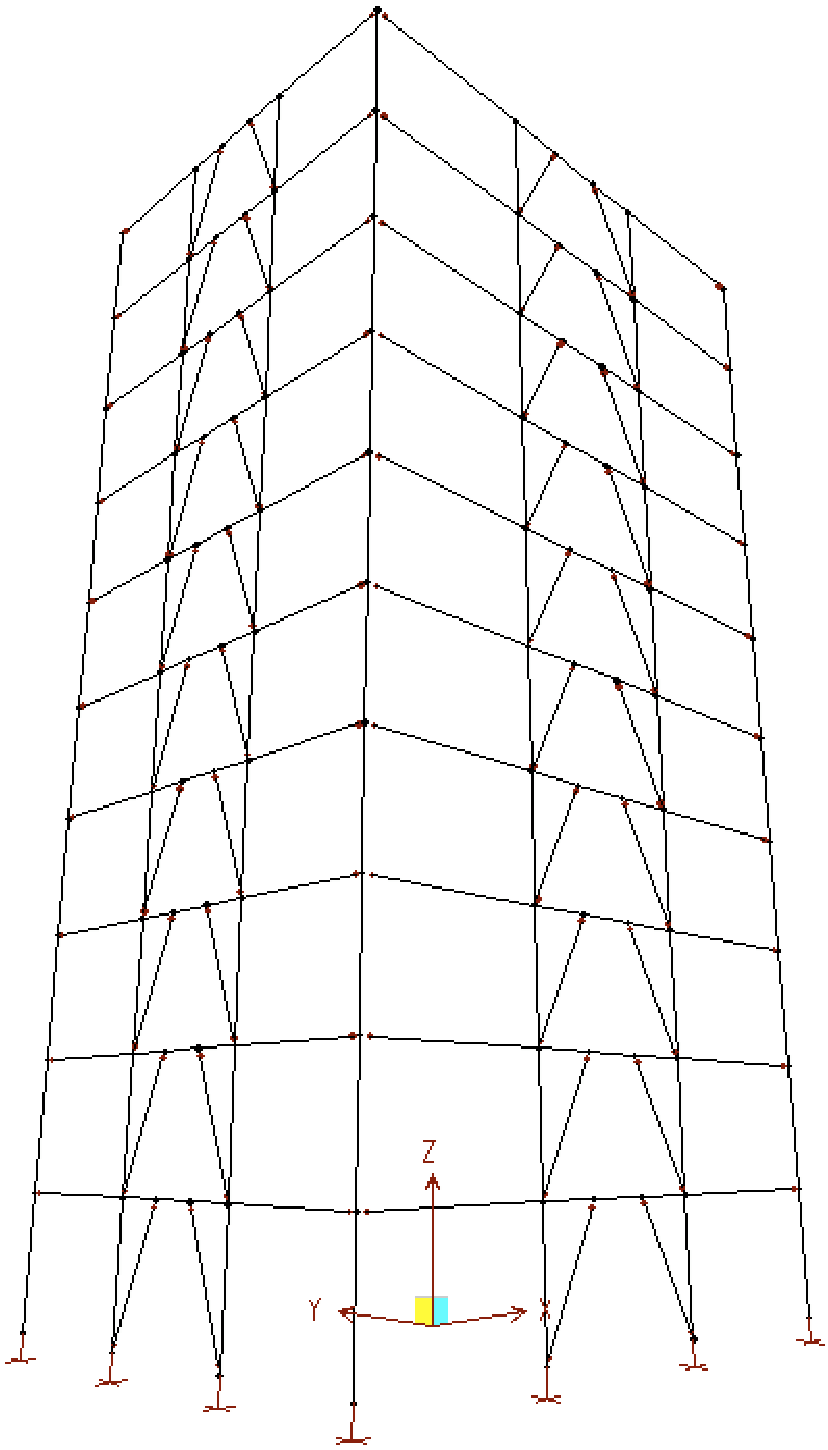

Figure A-2 10-Storey Model 3-D View

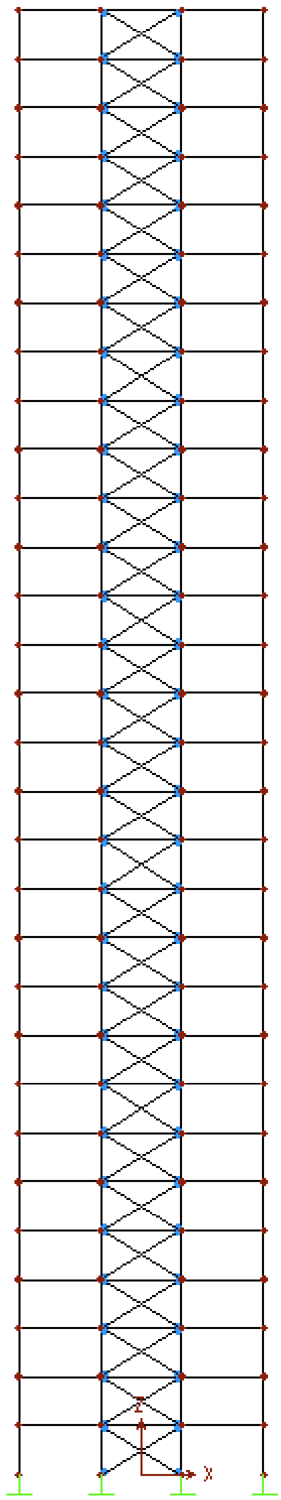

Figure A-3 30-Storey Model Side View

After creating the model, assigning initial sections to the members and defining the loads, the .s2k file is opened from OPTSTEEL (Figure A-4).

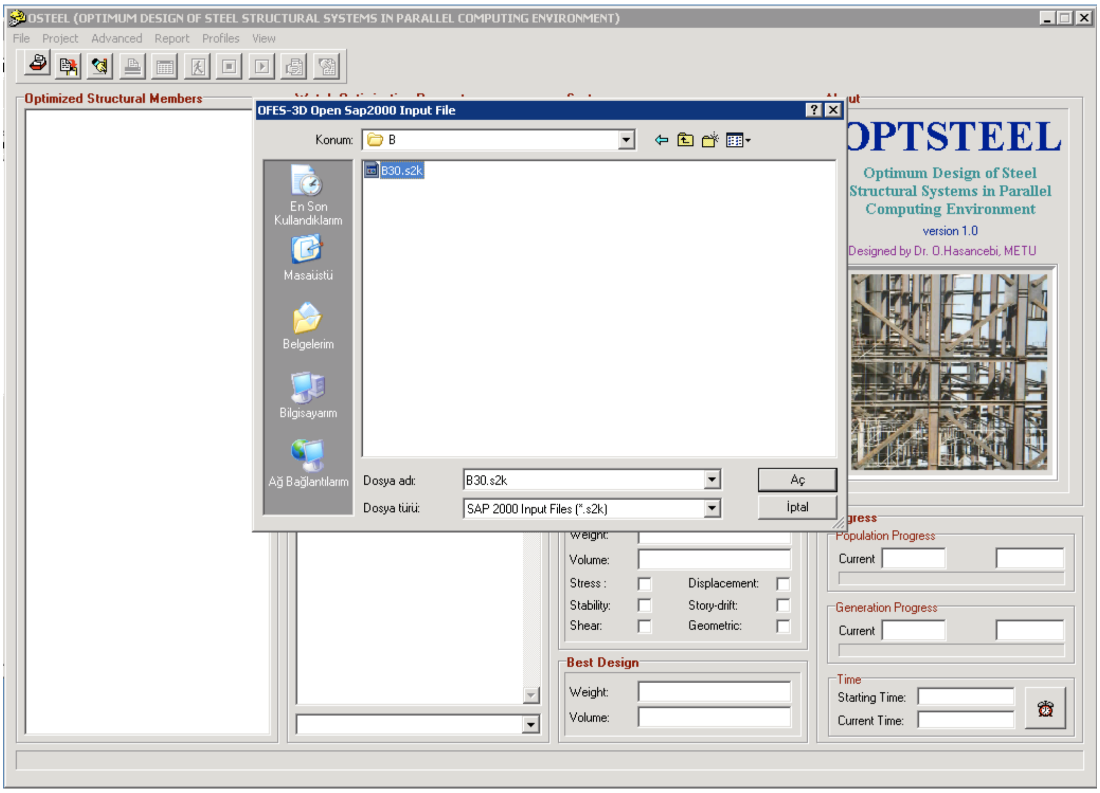

Figure A-4 Open SAP2000 Input File Screen of OPTSTEEL

Since this is an optimization software, the program needs some information of sections and their orientations from which it can selects and makes analysis with different combinations. From the define data section, the member profiles, their orientations and some other parameters can be defined. Here in Figure A-5 and Figure A-6, the member grouping file created before is recalled from "Define Data" section so that the members, that we want to be the same, are grouped and the software selects same steel section for all of the members of a group.

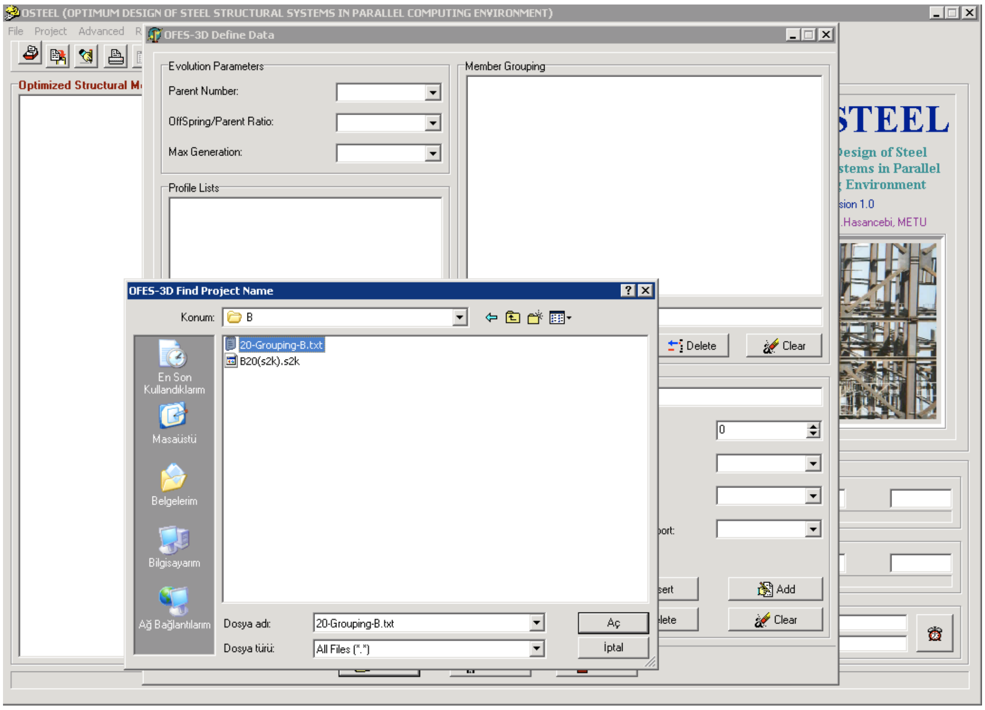

Figure A-5 “Open Grouping File” Screen of OPTSTEEL

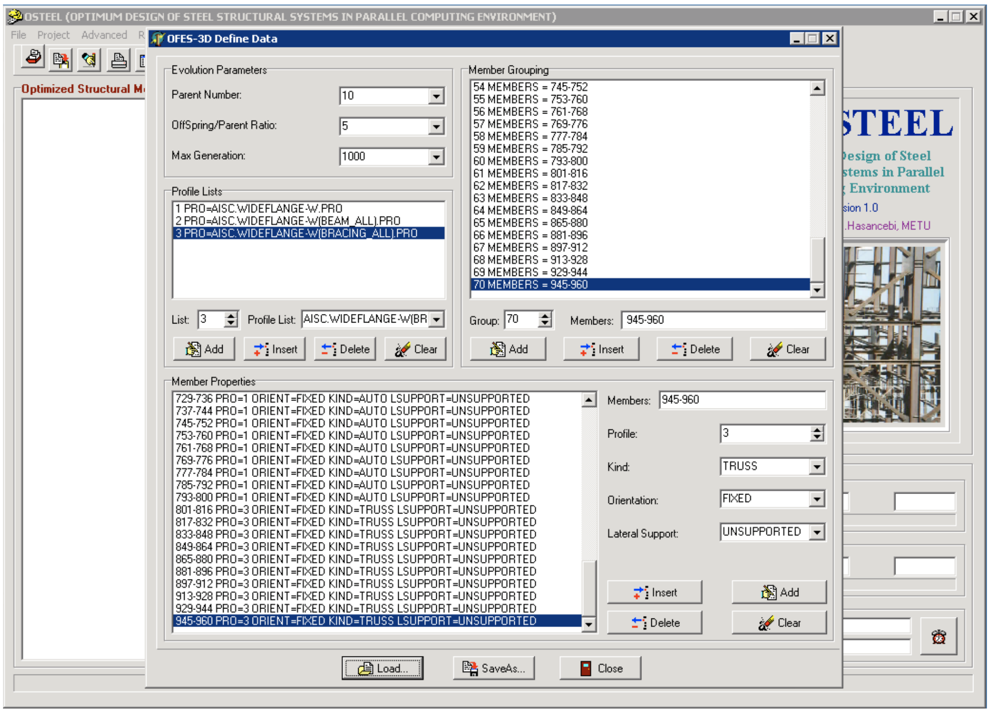

Figure A-6 "Member Grouping" Screen of OPTSTEEL

After that the constraints should be defined. The optimization parameters can be controlled here. The code, sidesway permission, displacement check, maximum displacements and drift constraints can be set here. Figure A-7 shows the constraints menu.

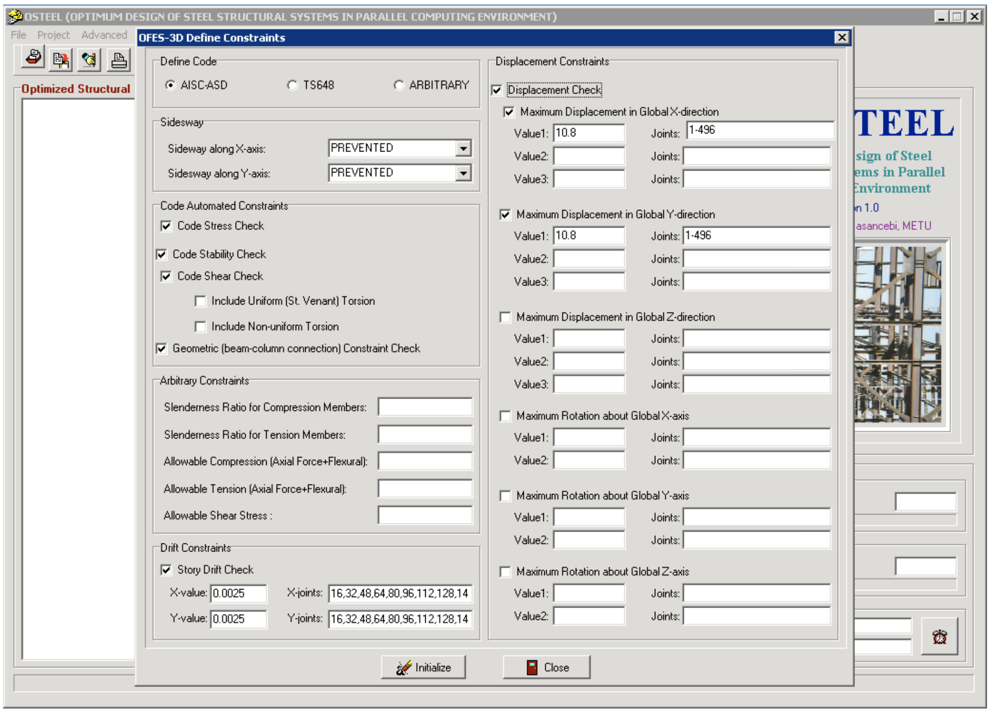

Figure A-7 "Constraints" Screen of OPTSTEEL

Finally, when all the parameters are set, the program is executed.

A.1 Sample Grouping File for OPTSTEEL

The sample grouping file for the 30-storey model (Model-B) can be seen below, as stated before there are 105 member groups in this model.

EVOLUTION
PN=10 RATIO=5 MAXGEN=1000

PROFILE LISTS
1 PRO=AISC.WIDEFLANGE-W.PRO
2 PRO=AISC.WIDEFLANGE-W(BEAM_ALL).PRO
3 PRO=AISC.WIDEFLANGE-W(BRACING_ALL).PRO

MEMBER PROPERTIES

1-24 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
25-48 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
49-72 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
73-96 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
97-120 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
121-144 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
145-168 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
169-192 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
193-216 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
217-240 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
241-264 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
265-288 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
289-312 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
313-336 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
337-360 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
361-384 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
385-408 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
409-432 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
433-456 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
457-480 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
481-504 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
505-528 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
529-552 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
553-576 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
577-600 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
601-624 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
625-648 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
649-672 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
673-696 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
697-720 PRO=2 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
721-728 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
729-736 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
737-744 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
745-752 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
753-760 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
761-768 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
769-776 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
777-784 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
785-792 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
793-800 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
801-808 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
809-816 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
817-824 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
825-832 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
833-840 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
841-848 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
849-856 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
857-864 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED

865-872 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
873-880 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
881-888 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
889-896 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
897-904 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
905-912 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
913-920 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
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945-952 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
953-960 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
961-968 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
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977-984 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
985-992 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
993-1000 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
1001-1008 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
1009-1016 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
1017-1024 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
1025-1032 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
1033-1040 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
1041-1048 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
1049-1056 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
1057-1064 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
1065-1072 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
1073-1080 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
1081-1088 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
1089-1096 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
1097-1104 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
1105-1112 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
1113-1120 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
1121-1128 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
1129-1136 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
1137-1144 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
1145-1152 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
1153-1160 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
1161-1168 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
1169-1176 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
1177-1184 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
1185-1192 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
1193-1200 PRO=1 ORIENT=FIXED KIND=AUTO LSUPPORT=UNSUPPORTED
1201-1216 PRO=3 ORIENT=FIXED KIND=TRUSS LSUPPORT=UNSUPPORTED
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1377-1392 PRO=3 ORIENT=FIXED KIND=TRUSS LSUPPORT=UNSUPPORTED
1393-1408 PRO=3 ORIENT=FIXED KIND=TRUSS LSUPPORT=UNSUPPORTED
1409-1424 PRO=3 ORIENT=FIXED KIND=TRUSS LSUPPORT=UNSUPPORTED
1425-1440 PRO=3 ORIENT=FIXED KIND=TRUSS LSUPPORT=UNSUPPORTED

GROUPS
1 MEMBERS = 1-24
2 MEMBERS = 25-48
3 MEMBERS = 49-72
4 MEMBERS = 73-96
5 MEMBERS = 97-120
6 MEMBERS = 121-144
7 MEMBERS = 145-168
8 MEMBERS = 169-192
9 MEMBERS = 193-216
10 MEMBERS = 217-240
11 MEMBERS = 241-264
12 MEMBERS = 265-288
13 MEMBERS = 289-312
14 MEMBERS = 313-336
15 MEMBERS = 337-360
16 MEMBERS = 361-384
17 MEMBERS = 385-408
18 MEMBERS = 409-432
19 MEMBERS = 433-456
20 MEMBERS = 457-480
21 MEMBERS = 481-504
22 MEMBERS = 505-528
23 MEMBERS = 529-552
24 MEMBERS = 553-576
25 MEMBERS = 577-600
26 MEMBERS = 601-624
27 MEMBERS = 625-648
28 MEMBERS = 649-672
29 MEMBERS = 673-696
30 MEMBERS = 697-720
31 MEMBERS = 721-728
32 MEMBERS = 729-736
33 MEMBERS = 737-744
34 MEMBERS = 745-752
35 MEMBERS = 753-760
36 MEMBERS = 761-768
37 MEMBERS = 769-776

38 MEMBERS = 777-784
39 MEMBERS = 785-792
40 MEMBERS = 793-800
41 MEMBERS = 801-808
42 MEMBERS = 809-816
43 MEMBERS = 817-824
44 MEMBERS = 825-832
45 MEMBERS = 833-840
46 MEMBERS = 841-848
47 MEMBERS = 849-856
48 MEMBERS = 857-864
49 MEMBERS = 865-872
50 MEMBERS = 873-880
51 MEMBERS = 881-888
52 MEMBERS = 889-896
53 MEMBERS = 897-904
54 MEMBERS = 905-912
55 MEMBERS = 913-920
56 MEMBERS = 921-928
57 MEMBERS = 929-936
58 MEMBERS = 937-944
59 MEMBERS = 945-952
60 MEMBERS = 953-960
61 MEMBERS = 961-968
62 MEMBERS = 969-976
63 MEMBERS = 977-984
64 MEMBERS = 985-992
65 MEMBERS = 993-1000
66 MEMBERS = 1001-1008
67 MEMBERS = 1009-1016
68 MEMBERS = 1017-1024
69 MEMBERS = 1025-1032
70 MEMBERS = 1033-1040
71 MEMBERS = 1041-1048
72 MEMBERS = 1049-1056
73 MEMBERS = 1057-1064
74 MEMBERS = 1065-1072
75 MEMBERS = 1073-1080
76 MEMBERS = 1081-1088
77 MEMBERS = 1089-1096
78 MEMBERS = 1097-1104
79 MEMBERS = 1105-1112
80 MEMBERS = 1113-1120
81 MEMBERS = 1121-1128
82 MEMBERS = 1129-1136
83 MEMBERS = 1137-1144
84 MEMBERS = 1145-1152
85 MEMBERS = 1153-1160

86 MEMBERS = 1161-1168
87 MEMBERS = 1169-1176
88 MEMBERS = 1177-1184
89 MEMBERS = 1185-1192
90 MEMBERS = 1193-1200
91 MEMBERS = 1201-1216
92 MEMBERS = 1217-1232
93 MEMBERS = 1233-1248
94 MEMBERS = 1249-1264
95 MEMBERS = 1265-1280
96 MEMBERS = 1281-1296
97 MEMBERS = 1297-1312
98 MEMBERS = 1313-1328
99 MEMBERS = 1329-1344
100 MEMBERS = 1345-1360
101 MEMBERS = 1361-1376
102 MEMBERS = 1377-1392
103 MEMBERS = 1393-1408
104 MEMBERS = 1409-1424
105 MEMBERS = 1425-1440

END