UNIVERSITY OF BAHRAIN

College of Engineering



Department of
Civil Engineering

Rapid Design Optimization of Rigidly Connected 3-Dimensional Steel Frames Under LRFD - AISC Design Code Provisions

A Thesis Submitted in Partial Fulfilment of the Requirements for the M.Sc. Degree in Civil Engineering

Submitted by

Ahmed Tahsin Ahmed Channaa

University ID: 20105490

Supervised by

Dr. Semih Erhan

Assistant Professor

University of Bahrain

Dr. Mona Abdullatif

Assistant Professor

University of Bahrain

Kingdom of Bahrain

February, 2023

UNIVERSITY OF BAHRAIN

College of Engineering

Department of Civil Engineering

Rapid Design Optimization of Rigidly Connected 3-Dimensional Steel Frames Under LRFD - AISC Design Code Provisions

A Thesis Submitted in Partial Fulfilment of the Requirements for the M.Sc. Degree in Civil Engineering

Submitted by

Ahmed Tahsin Ahmed Channaa

University ID: 20105490

Supervised by

Dr. Semih Erhan

Assistant Professor

University of Bahrain

Dr. Mona Abdullatif

Assistant Professor

University of Bahrain

Kingdom of Bahrain

February, 2023

University of Bahrain
Deanship of Graduate Studies
And scientific Research

جامعة البحرين
عمادة الدراسات العليا والبحث العلمي

Thesis Discussion Committee Decision

Student Name: **Ahmed Tahsin Ahmed Channaa**	Student ID: **20105490**
Department: **Civil Engineering**	College: **Engineering**
Program: **Masters in Civil Engineering**	
Thesis Title: **Rapid Design Optimization of Rigidly Connected 3-Dimensional Steel Frames Under LRFD-AISC Design Code Provisions**	
Supervisor Name: **Dr. Semih ERHAN & Dr. Mona Abdullatif**	Student GPA: **4.00**

The Discussion Committee met on 26-Feb-23 In Conference Room in Building 29, Isa Town. Upon discussing the student whose details are stated above in public, the discussion committee recommends the following:

The student is awarded a grade A in the thesis subject.

☐ Without making any changes to the thesis.

☒ After making the modifications attached with this decision.

Discussion Committee Members Approval:

Name		Signature
1. Dr. Semih Erhan & Dr. Mona Abdullatif	(Advisor)	
2. Prof. Dr. Mustafa Aytekin	(Internal Examiner)	
3. Assoc. Prof. Serdar Carbas	(External Examiner)	

Approved at the University Council 6th meeting, 2013
In accordance with Decision No. 736/2013 dated 14/3/2013
Modified by University Council Decision No. 2043/2013 dated 20/11/2013.

Abstract

In this thesis, the design optimization of rigidly connected steel space frames is carried out under the American Institute of Steel Construction – Load and Resistance Factor Design (LRFD-AISC) design code provisions. The algorithm developed is a combination of optimality criteria method, which is a deterministic method, and Beetle Antenna Search (BAS) algorithm, which is a metaheuristic method. The design code provisions are satisfied by the algorithm, while minimizing the total weight of the steel space frame. The sections of the frame members are selected from the list of available American W-sections. The modeling of the steel frames is done using SAP2000 software. The design code provisions, as well as the optimization algorithm formulations are coded in MATLAB. The interface between SAP2000 and MATLAB is achieved using the Open Application Programming Interface (OAPI). The design optimization results are demonstrated by applying the developed algorithm to a single storey single bay steel space frame, as well as a five storey two bay steel space frame that was selected from a past research paper. In this research, the number of iterations (structural analysis) is reduced drastically as compared to other algorithms that are based on metaheuristic methods only. In other words, the algorithm is shown to be efficient in obtaining rapid design optimization of rigidly connected steel space frames.

Dedication

To my father and mother, who always supported me and believed in me.

To my wife and daughter, who motivated me.

To my brother and sisters, who encouraged me.

To my family and friends, who were there for me.

Acknowledgements

My greatest thanks and gratitude go to my father and mother, who were the main reason I got enrolled in the MSc program, in the first place. My father always believed in me and supported me. He encouraged me and still encourages me to continue the Postgraduate studies till PhD. My mother had supported me with her love, affection, prayers, and all possible means. She too is encouraging me to continue my studies till PhD level. My parents are the reason for all the success and achievements in my life.

Secondly, I would like to thank my wife for her support and encouragement during my enrollment in the program. Our first child, Layan, is a gift from Allah. She was born just a few months before the final submission, and was a motivation to complete this work, as well.

Special thanks go to my previous supervisor, Prof. Dr. Mehmet Polat Saka, who supervised my thesis for more than a year. I was honored to be his student in five MSc level courses. He never hesitated to provide me with the necessary support and guidance. He always believed in me and his words were really encouraging and motivating. He was always happy to share his great knowledge and experience with his students.

Many thanks also go to the Head of Civil Engineering Department at University of Bahrain, Dr. Nuha Al Zayani, and my current supervisors, Dr. Semih Erhan and Dr. Mona Abdullatif, for all the support, guidance, and encouragement they provided me to complete the thesis successfully.

Table of Contents

Defense Committee's Approval ..... A
Abstract ..... B
Dedication C
Acknowledgements ..... D
Table of Contents ..... E
List of Tables ..... H
List of Figures I
List of Abbreviations ..... K
Chapter 1 Introduction 2
1.1. Optimization 2
1.2. Structural Optimization 3
1.3. Research Problem 4
1.4. Research Questions 4
1.5. Research Objectives 5
1.6. Research Significance 5
1.7. Research Scope 5
1.8. Research Outline 6
Chapter 2 Theoretical Framework and Literature Review 8
2.1. Introduction 8
2.2. Design of Steel Frames 9
2.3. Design Optimization of Steel Frames 10
2.4. Elements of Steel Frames Design Optimization 11
2.4.1. Design Variables 11
2.4.2. Objective Function 11
2.4.3. Constraints 12
2.5. Literature Review for the Optimum Design of Steel Frames 13
2.5.1. Deterministic Methods for Optimum Design of Steel Frames. 14

2.5.2. Metaheuristic Methods for Optimum Design of Steel Frames 17
2.6. Summary 21
Chapter 3 Optimum Design of Steel Space Frames 24
3.1. Objective Function 24
3.2. Design Variables 24
3.3. Design Constraints 25
3.3.1. Strength Constraints 25
3.3.1.1. Combined Axial Force and Bending 25
3.3.1.2. Design Tensile Strength 26
3.3.1.3. Design Compressive Strength 27
3.3.1.4. Design Flexural Strength about Major Axis 29
3.3.1.5. Design Flexural Strength about Minor Axis 33
3.3.2. Displacement Constraints 34
3.3.2.1. Deflection Constraints 34
3.3.2.2. Drift Constraints 34
3.3.3. Loading and Load Combinations 35
3.4. Summary 36
Chapter 4 Methods and Procedures 38
4.1. Introduction 38
4.2. SAP2000 Modelling 38
4.2.1. Steel Space Frame Model 38
4.2.2. Steel Space Frame Model Loading 41
4.2.3. SAP2000 Analysis Results 44
4.3. Open Application Programming Interface (OAPI) 46
4.4. Optimality Criteria Method 48
4.4.1. Mathematical Formulation 48
4.5. Beetle Antenna Search Algorithm 50
4.6. Combined Optimization Algorithm 53
Chapter 5 Results and Discussion 57

5.1. Introduction.....57
5.2. Single Storey Single Bay Steel Space Frame.....57
5.2.1. Optimization for Displacements Constraints Only.....60
5.2.2. Optimization for Strength Constraints Only.....62
5.2.3. Overall Design Optimization of Single Storey Frame.....64
5.3. Five Storey Steel Space Frame.....66
Chapter 6 Conclusion and Recommendations.....70
6.1. Research Findings and Conclusion.....70
6.2. Recommendations for Further Study.....71
References.....73
Appendix A W-sections pool.....
Appendix B OAPI Applications.....
الملخص.....

List of Tables

Table 3.1. Displacement limitations of steel frames. 35
Table 4.1. Steel space frame member groups in SAP2000 model 41
Table 4.2. Beam gravity loading of the 105-member regular steel space frame (Carbas, 2016) 41
Table 4.3. OAPI functions used for Design Optimization of Steel Space Frame 47
Table 4.4. Parameters of Steel Space Frame Design Optimization Algorithm 55
Table 5.1. Results of Single Storey Frame Design Optimization to Drift Constraints 61
Table 5.2. Algorithm Parameters of Drift Constraints for Single Storey Frame 61
Table 5.3. Results of Single Storey Frame Optimization to Strength Constraints 63
Table 5.4. Algorithm Parameters of Strength Constraints for Single Storey Frame 63
Table 5.5. Results of Single Storey Frame Design Optimization.....65
Table 5.6. Design Optimization Algorithm Parameters for Single Storey Frame 65
Table 5.7. Results of Five Storey Frame Design Optimization.....66
Table 5.8. Design Optimization Algorithm Parameters for Five Storey Frame 67

List of Figures

Figure 2.1 Steel Space Frame (Thearchitectsdiary.com, 2020).....8
Figure 2.2 Beam column geometric constraints (Carbas, 2016) 13
Figure 3.1. Standard rolled shapes (Williams, 2011) 25
Figure 3.2. Standard column curve (Williams, 2011) 29
Figure 3.3. Nominal moments and unbraced length (Williams, 2011) 31
Figure 4.1. 105-member regular steel space frame: (a) 3D view, (b) plan view (Carbas, 2016).....39
Figure 4.2. 105-member steel space frame SAP2000 model: (a) 3D view, (b) Plan view.....40
Figure 4.3. Dead load (kN/m) assigned on beams of the steel space frame 42
Figure 4.4. Wind load in X direction (Auto SAP2000 assigned).....43
Figure 4.5. Wind load in Y direction (Auto SAP2000 assigned).....43
Figure 4.6. Diagram of bending moment about major axis under ULS-1 load combination.....44
Figure 4.7. Analysis results of a typical frame member.....45
Figure 4.8. Flowchart of Steel Space Frame Design Optimization Algorithm.....54
Figure 5.1. Single Storey Single Bay Steel Space Frame.....58
Figure 5.2. Dead Load (kN/m) Assigned to Beam Members of Single Storey Frame.....58
Figure 5.3. Live Load (kN/m) Assigned to Beam Members of Single Storey Frame 59
Figure 5.4. Wind Load (kN) Assigned to The Single Storey Frame Joints 59

Figure 5.5. Single Storey Frame Optimization According to Drift Constraints 60
Figure 5.6. Single Storey Frame Optimization According to Strength Constraints 62
Figure 5.7. Single Storey Frame Overall Design Optimization 64
Figure 5.8. Five Storey Frame Overall Design Optimization 67

List of Abbreviations

2D	Two Dimensional
3D	Three Dimensional
ABC	Artificial Bee Colony
AISC	American Institute of Steel Construction
ASD	Allowable Stress Design
ASTM	American Society for Testing and Materials
BAS	Beetle Antenna Search
CSI	Computers and Structures Incorporation
DE	Differential Evolution
EFFA	Enhanced Firefly Algorithm
FFA	Firefly Algorithm
LRFD	Load and Resistance Factor Design
OAPI	Open Application Programming Interface
UB	Universal Beam
UC	Universal Column
UDL	Uniformly Distributed Load

Chapter 1

Introduction

Chapter 1 Introduction

1.1. Optimization

Optimization have become a field of interest and research, as many businesses, industries and factories are aiming to maximize their production and minimize their costs such as manpower, material, and energy consumption. There is an urge for factories to cut their costs, to be able to survive in a highly competent market and increase their profit margins, but without compromising the product quality.

An optimization problem is a problem in which certain parameters, called "design variables", need to be determined to achieve the best measurable performance. In the mathematical formulation of an optimization problem, a certain function called "objective function" is either maximized or minimized under certain conditions called "constraints". Both objective function and constraints are expressed as functions of design variables. The general form of an optimization problem is as follows:

Minimize

$$f(\boldsymbol{x}) \tag{1.1}$$

Subject to

$$g_i(\boldsymbol{x}) \leq 0 \ , \ i = 1, \dots, ng \tag{1.2}$$

$$h_j(\boldsymbol{x}) = 0 \ , \ j = 1, \dots, ne \tag{1.3}$$

Where $\boldsymbol{x}$ is the design variable vector of size n, $f(\boldsymbol{x})$ is the objective function, and $g_i(\boldsymbol{x})$ and $h_j(\boldsymbol{x})$ are inequality and equality constraints. ng is the total number of inequality constraints and ne is the total number of equality constraints.

1.2. Structural Optimization

Applications of optimization in Structural Engineering are vast and of great significance. The aim of a Structural Engineer is to design a structure that is safe and yet economical. For instance, different Structural Engineers, may end up with endless number of design solutions that are safe and fulfill design code requirements, yet with varying costs for building the structure. Structural Optimization techniques aim to find the best possible design solution that fulfills the strength and serviceability requirements, as well as satisfies the economical aspect. In other words, structural optimization aims to reach to an optimum design.

The formulation of the steel frame design according to design codes is an optimization problem. The objective function is minimization of the steel frame weight. The design variables are the sections of the steel frame members, namely beams and columns. The constraints are the strength and serviceability limitations.

Solution of the optimum design problem of steel structures can be achieved using Metaheuristic algorithms. These algorithms mimic the natural phenomena such as Genetic Algorithms, Simulated Annealing, Ant Colony Optimization, Particle Swarm Optimizer, and Harmony Search Method. These techniques are also called stochastic solution techniques. Alternatively, optimization of steel structures can be based on deterministic methods such as optimality criteria method and gradient method (Saka & Geem, 2013).

Deterministic methods are based on mathematical relations and computation of the function derivative, they require less time to reach to an optimum solution and guarantee a global optimal solution. On the other hand, metaheuristic methods do not require the function derivate, they

require a long time to find optimum solution, and the global optimal solution is not guaranteed unless the running time is infinite. The major disadvantage of these methods is that they need large number of structural analysis during the optimum design cycles, which is computationally very expensive.

The aim of this study is to achieve a design solution for steel space frames within a reasonably short time, making it practical to be used in Engineering design offices. The intention of the study is to use combination of both deterministic and metaheuristic methods, to benefit from the advantages of both approaches, aiming to produce an algorithm which finds the optimum design of steel structures rapidly. The serviceability limitations are handled with optimality criteria method, while the strength constraints are treated by the beetle antenna search algorithm.

1.3. Research Problem

Structural optimization of steel space frames intends to minimize the overall weight of the structure, and yet satisfy the design code provisions of the American Institute of Steel Construction, Load and Resistance Factor Design (LRFD-AISC). The variable in this case is the cross section of the steel profiles available in the practice. The aim is to determine the appropriate steel profiles for the members of the space frame such that the weight or construction cost of the frame is minimized, while the LRFD-AISC design code provisions are satisfied.

1.4. Research Questions

The research questions are as follows:

i) What is the optimum cross section of each group of steel members in the space frame?

ii) What is the minimum weight of the space frame that can be achieved?

iii) How many design cycles are needed to achieve this solution?

1.5. Research Objectives

The research objectives are as follows:

i) Develop a structural optimization algorithm for three-dimensional (3D) steel frames with rigid beam column connections.

ii) Determine the most economical steel profiles for the members of the frame, so that the response of the frame under the external loads satisfies the design code provisions (LRFD-AISC).

iii) Minimize the number of iterations needed to reach to the optimum design solution.

1.6. Research Significance

The significance of this research is that the design optimization of steel space frame, minimizes the amount of steel material needed for a particular structure, thus reducing the cost and the environmental impact of steel production.

Moreover, a rapid design optimization, with a reasonable number of iterations, makes it more practical to apply this technique in Engineering design offices.

1.7. Research Scope

This research aims to develop an optimum design algorithm for rigidly connected steel space frames. The objective is to minimize the weight of the structure, while satisfying the LRFD-AISC design code provisions. The frame is subjected to dead loads, live loads, snow loads and wind loads.

The 3D modeling has been done using SAP2000 software, and the algorithm has been coded in MATLAB. The integration of these two software has been done using Open Application Programming Interface (OAPI), where the analysis results were obtained from SAP2000 and fed into the MATLAB routine, and this is carried out iteratively until the optimum design of the space frame was obtained.

For the strength constraint, the metaheuristic approach was adopted, namely the Beetle Antenna Search (BAS) algorithm and the displacement (serviceability) constraints are handled by a deterministic approach, namely the optimality criteria method.

The design variables were the W-sections available in the discrete list of AISC (wide flanged sections) that had to be selected for the beams and columns of a space frame.

1.8. Research Outline

The thesis consists of six chapters. The first chapter is an introduction to the research topic, which is design optimization. The second chapter is a literature review on the optimization of steel frames. The third chapter describes the optimum design of steel frames. The fourth chapter describes the methods and procedures used in the optimization process. The fifth chapter is a discussion of the obtained results. Finally, the sixth chapter is a conclusion of the research and recommendation for improvement and further study.

Chapter 2

Theoretical Framework and Literature Review

Chapter 2 Theoretical Framework and Literature Review

2.1. Introduction

Steel structures have become more widespread in the recent days. The use of structural steel is not limited to high rise buildings anymore. The durability and strength of steel, coupled with the ease and speed of erection and assembling on site after purchasing the prefabricated steel profiles, has made steel a preferred option for a Structural Engineer's consideration at the structural design stage. Moreover, buildings made up of steel frames can allow for greater spans, thus reducing the number of columns and increasing the usable space in a building. Steel has favorable mechanical properties such as ductility and weldability. Another merit of steel structures is the fact that steel is a recyclable material, making it an environmentally friendly construction material. Figure 2.1 shows a typical steel space frame (3D frame) building.

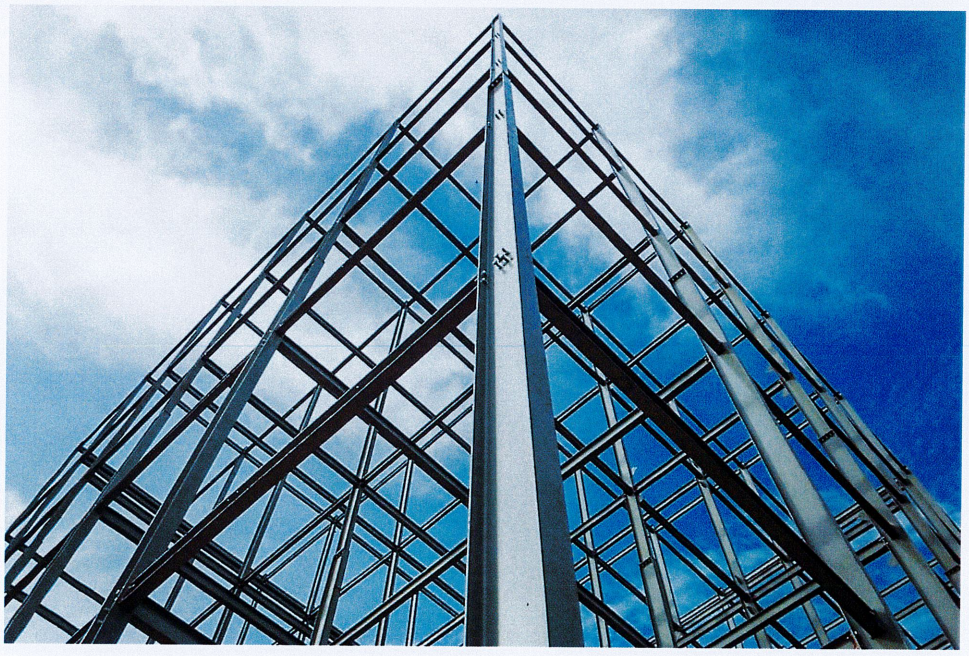


Figure 2.1 Steel Space Frame (Thearchitectsdiary.com, 2020)

2.2. Design of Steel Frames

Steel structures are designed with respect to international standards such as:

i) British Standard – Structural use of steelwork in building (BS 5950).
ii) Eurocode 3: Design of Steel Structure.
iii) American Institute of Steel Construction – Specification for Structural Steel Buildings (AISC-360).

The American Institute of Steel Construction (AISC-360) is one of the most widespread specifications applied for structural steel design. The code describes two approaches as follows (Williams, 2011):

1. Allowable Stress Design (ASD): this is a traditional method, which aims to ensure that the structure is capable to withstand the service loads (dead loads and live loads) safely. Dead loads consist of self-weight of the structure, permanent finishes, and fittings, while the live load consists of movable objects, occupants, etc. The stresses developed in the structure due to these loads shall not exceed a specified allowable stress.
2. Load and Resistance Factor Design (LRFD): in this approach the service loads are increased by a factor of safety (load factor), while the resistance (capacity) of the steel members is reduced by a resistance factor.

The latter approach (LRFD-AISC) is more widespread and is believed to provide more accurate results and more economical designs as compared to the ASD approach.

2.3. Design Optimization of Steel Frames

Structural Engineers must select the sections of steel members from a list of practically available steel sections. The possible combinations are numerous in this type of combinatorial problems. For example, if the steel space frame members are divided into four groups and the Engineer is to select from a list of 283 available steel sections, then the number of possible combinations would be 283^4, which equals 6.41 billion possibilities. The time it would take an Engineer to try all the possible combinations and analyze the structure with them is infinite. Therefore, the significance of structural design optimization has emerged accordingly.

Structural optimization of steel frames has been a research interest for Engineers in the past few decades. Different approaches have been used with a variety of structural optimization algorithms. However, these methods can be classified into two broad categories as follows:

1. Deterministic algorithms

Deterministic algorithms are the traditional methods used for optimization. They are also called mathematical programming techniques. They are based on solid mathematical formulations and principles and do not consider any uncertainties. They require the computation of the objective function derivative. Some examples, include optimality criteria method and gradient methods (Saka & Geem, 2013).

2. Metaheuristic algorithms

Metaheuristic algorithms do not require the computation of the objective function derivative. Instead, these are based on behavior of natural phenomena. The values of design variables are selected in a random manner and then the constraints are checked, until the objective function is

minimized. Some examples are Genetic algorithm, Evolution Strategies, Particle Swarm Optimizer and Harmony Search algorithm (Saka & Geem, 2013).

2.4. Elements of Steel Frames Design Optimization

The elements of steel frames design optimization are the design variables, the objective function, and the constraints.

2.4.1. Design Variables

The design variables are the parameters that the change of their values affect the value of the objective function. The design variable could be one or more depending on nature of the design problem and the intent of the designer. The design variable may be the cross-sectional area of the steel frame member. It could be the type of beam to column connection, whether rigid or pinned. It could also be the joint coordinates or even the existence of members, in other words whether the existence or absence of a particular member is more economical.

The design variables for steel space frames are usually discrete. For example, if the design variable is section of the steel member, which must be selected from a list of available standard W-sections, which makes the design variable discrete and not continuous.

2.4.2. Objective Function

The objective function determines the quality of the optimization solution. Common Engineering objective functions aim to minimize the cost. One of the main factors that affect the cost is the material quantity, therefore, the common steel design optimization objective function is to minimize the weight of the structure with an intention to minimize the overall construction cost. However, other factors affect the overall cost of

construction, including material transportation, material availability, labor charges, construction duration, and maintenance costs of the selected construction material. All these can be included in the objective function if the designer wishes to do so.

2.4.3. Constraints

The constraints are the conditions that need to be satisfied, while evaluating the objective function. The optimization problem with these conditions is called a constrained optimization problem, while the optimization problems without any limitations are called unconstrained optimization problems. One of the approaches used to convert a constrained optimization problem into an unconstrained one is to use a penalty function. Constraints in steel design optimization may include the following:

1. Strength Constraints: The steel design code provisions such as that of LRFD-AISC makes it necessary to provide certain strength to the beams and columns of a steel frame so that the frame does not fail under the action of external loads. These requirements specified by LRFD-AISC are the constraints of steel frame design optimization. It is a must in the design of steel frame the strength requirements of beams and columns subjected axial force and biaxial bending are satisfied. These conditions are called the strength constraints.
2. Displacement Constraints: The maximum allowable top storey drift and maximum allowable beam deflections shall not be exceeded by the frame members under the service load case. This is serviceability requirements imposed by the design code. These are called the displacement constraints.
3. Geometric Constraints: Geometric constraints are constructability requirements that allow for a proper geometric relation in beam-column

connections and column-column connections, such that the beams and columns can be appropriately connected to each other, and their dimensions fit to each other. For example, the width of the beam shall not exceed the width of the column, when the beam is connected to the flange of the column. Moreover, the width of the beam shall not exceed the clear depth between the column flanges when the beam is connected to the web of the column. Figure 2.2 shows the beam to column geometric constraints. Other constraints also include minimum size constraints.

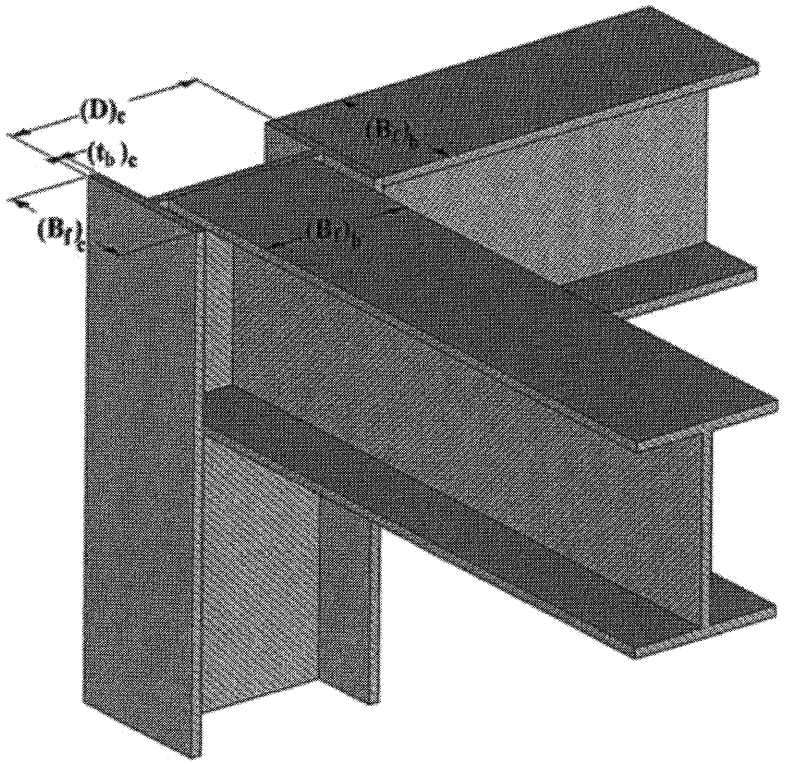


Figure 2.2 Beam column geometric constraints (Carbas, 2016)

2.5. Literature Review for the Optimum Design of Steel Frames

The design optimization of steel frames has been of a great interest to researchers for the past few decades. Several research papers related to

optimum design of steel frames have been published in the literature. Some of the articles have considered two dimensional (2D) frames or plane frames, while others have expanded their study to cover 3D frames or space frames. Some researchers used deterministic solution techniques, while others have used metaheuristic solution techniques. A brief description of the reviewed papers is given in this section, according to the optimization approach, while following a chronological order.

2.5.1. Deterministic Methods for Optimum Design of Steel Frames

Saka (1991) had published a paper on the use of optimality criteria method to optimize the design of 2D steel frames, while considering the AISC design code provisions. The design variables were taken as the cross-sectional areas of steel members as continuous variables. To reduce the number of variables, other sectional properties, such as second moment of area and sectional modulus were expressed in terms of cross-sectional area. The objective function aimed to minimize the total weight of the steel plane frame. The design constraints were displacement constraints and stress constraints. They are included in the mathematical formulation as the design provisions specified by ASD-AISC.

An optimality criteria algorithm was used for the displacement constraints. The required cross-sectional area for each group of steel members was computed using the concept of virtual work method. This cross-sectional area was considered as the lower bound to fulfill the displacement constraint.

For the stress constraints, the ASD-AISC approach was adopted. The design code's combined stress inequalities, which include in-plane and lateral buckling of members, were reduced into nonlinear equations. These

equations included one variable only, which is the cross-sectional area of steel members. The solution of these equations provided the lower bound of design variable for the stress constraints.

The constraint with the greatest lower bound value of the design variable, was considered as the dominant constraint and its lower bound was used to decide the value of area variable. This procedure was done in an iterative process, simultaneously with the frame analysis.

This optimization algorithm was applied to a portal frame, pitch roof frame, six storey two bay frame, and ten storey three bay frame. The beam-column connections in the multi storey frames were considered as rigid as a first case, and they were considered as pinned connections in a second case. However, in the second case lateral bracing members have been introduced.

It was observed that rigidly connected frames yielded less overall weight, when subjected to gravity loads only. However, the frames with simple beam column connections and bracing, produced better design and lighter weight when subjected to lateral loads as well.

The conclusion was that the method used was effective and it yielded the optimum design solution within a minimal number of iterations. The optimum design solution was obtained in less than ten iterations in case of dominance of stress constraints. However, in case of dominance of displacements constraints the number of iterations needed was slightly higher.

Later, Saka and Kameshki (1998a) had applied the same concept, but with consideration of the nonlinear response of the steel plane frame. The constraints considered were sway constraints and combined stress limitations, as per the code provisions of ASD-AISC. The design variable

was the cross-sectional areas of the steel frame members. The objective function was the minimum weight of the steel frame. The optimality criteria approach was used to handle the sway limitations, while the combined stress constraints were reduced into nonlinear equations of design variables. The algorithm was applied to three different steel plane frames, as design examples. Two cases were considered; one was with lateral buckling constraints and the other was without the lateral buckling constraints. It was shown that convergence could be achieved in no more than 20 iterations. It was also shown that the consideration of lateral buckling constraints yielded a higher overall weight of the frame.

Moreover, Saka and Kameshki (1998b) had applied the optimality criteria approach to limit the frame sway. This was paired with the fully stressed design concept according to ASD-LRFD to optimize the design of rigidly connected 3D steel domes. The design considered nonlinear elastic framed domes. Minimization of the overall weight of the frame was the objective function. The design variable was the cross-sectional area of the steel dome members. The constraints considered were the sway constraints and stress constraints. The combined stress constraints were reduced into nonlinear equations of design variables. The second order effects have been taken into consideration; the stability functions for 3D beam-columns were used to obtain the P-δ effect in the frame. It was shown that the consideration of nonlinear elastic frame produced more realistic results. The approach used yielded more economical design.

Al-Mosawi and Saka (2000) used the optimality criteria method to obtain the optimum shape design of cold formed thin-walled steel beams. The objective function was to minimize the weight of the steel beam. The design variables were the cross-sectional dimensions such as width, depth, and thickness. The design constraints were displacements constraints and

stress constraints. It was shown that the method is effective to obtain the optimum design solution, even though the problem was found to be highly nonlinear.

2.5.2. Metaheuristic Methods for Optimum Design of Steel Frames

Camp and Bichon (2004) developed discrete design optimization algorithm for steel space trusses based on Ant Colony optimization technique. The total weight of space truss was minimized, as an objective function. The design constraints considered were stress and deflection limitations. The method was found to be efficient by comparing the obtained results with genetic algorithm and gradient based optimization methods.

Later, Camp, Bichon, and Stovall (2005) they extended the work to rigid steel frames. The design constraints were serviceability and strength constraints as per the code provisions of LRFD-AISC. The results were compared with those obtained by Saka and Kameshki (1998a), which is mentioned in *cl.* 2.5.1.

Saka (2009) optimized the design of 2D steel sway frames using harmony search algorithm, which is a stochastic search technique. This algorithm mimics the behavior of a musician who searches for a better harmony state. The objective function was minimization of the overall weight of frame. The design variables were the sections of steel beams and columns, which had to be selected from the standard list of British sections. The section pool included 64 Universal Beam (UB) sections and 32 Universal Column (UC) sections. The constraints considered were the design code provisions of the code British steel design code (BS 5950). The method adopted does not require a list of candidate solutions nor does it need an initial value for the design variable. The algorithm was applied to a couple of design

examples. It was shown that the method was robust and efficient in obtaining the optimum design of steel plane frames. However, the number of iterations needed was high; approximately 25,000 iterations.

Hasancebi, Carbas, Dogan, Erdal and Saka (2010) have presented a comparison of various non-deterministic (metaheuristic) techniques used in the design optimization of real size steel space frames. The study considered seven different algorithms. The algorithms considered in this comparison were Simulated Annealing, Evolution Strategies, Genetic Algorithms, Particle Swarm Optimizer, Tabu Search, Ant Colony Optimization, and Harmony Search Algorithm. The strength constraints were according to the ASD-LRFD design code provisions. Other constraints considered were the displacement constraints and geometric constraints. Minimization of the weight of the frame was the objective function, while the design variable was the section of the steel members that were selected from the standard list of W-sections. The maximum number of iterations used was 50,000 iterations. It was concluded that Evolution Strategies was the best approach among the seven non-deterministic methods considered in the study. This approach demonstrated a rapid convergence to the optimum design solution. Simulated Annealing was found to be the second-best method, as it showed slower, yet gradual convergence as compared to the Evolution Strategies approach.

Dogan and Saka (2012) have optimized the design of unbraced plane steel frames using Particle Swarm Optimization. This technique simulates the behavior of birds and fishes. The objective function was minimization of the steel frame weight. The strength constraints considered were imposed in accordance with LRFD-AISC. Displacement constraints and geometric constraints were also considered in the design optimization. The variable

was the sections assigned to beams and columns that were selected from a list of 272 W-sections. The algorithm generated continuous values for the design variables, which had to be rounded up to the nearest discrete value obtained from the steel section list. The method was found to be effective and robust in the design optimization of steel plane frames. The design examples presented, had shown that the Particle Swarm Optimization yielded a lighter frame as compared to other metaheuristic techniques considered in the study, namely Simulated Annealing and Genetic Algorithm.

Aydogdu, Akin and Saka (2016) optimized the design of real-world steel frames using an enhanced Artificial Bee Colony (ABC) algorithm. This algorithm mimics the natural foraging behavior of honeybees. The objective was minimization of the weight of the frame. The design constraints were the code provisions of LRFD-AISC including strength and displacement constraints. For strength constraints, the inequalities for combined axial force and biaxial bending have been considered for the steel frame elements, while for the displacement constraints the top storey drift and beam deflections had to be limited as per the code provisions. The design variable was the cross section of the steel profiles that had to be selected from the standard list of American W-sections. The maximum number of iterations was 50,000 for two design examples (132 members' and 428 members' space frames) and 75,000 for a third design example (1,024 members' space frame). It was concluded that the enhanced ABC algorithm was robust and efficient to obtain the optimum design solution of steel space frames. The enhanced ABC algorithm yielded a total frame weight that was 25% lighter than that of the standard ABC algorithm.

Carbas (2016) optimized the design of steel space frames using an enhanced firefly algorithm (EFFA). The design variable was the section of

the steel frame members, to be selected from the standard W-sections list. The objective was to minimize the overall weight of the frame. The constraints were the strength and displacement constraints according to LRFD-AISC design code provisions, as well as the geometric constraints. This was applied to two design examples (105-member regular steel space frame and 568-member unbraced steel space frame). The obtained minimum weights in the first and second design examples, were 38% and 25% lighter than that obtained by the standard firefly algorithm (FFA), respectively. The standard FFA was found to have a demerit of stagnation in a local optimum.

Latif and Saka (2019) have optimized the design of tied arch bridges using enhanced ABC algorithm. OAPI was used for obtaining the analysis results in each iteration. The objective function was minimization of the total weight of bridge. The design variables were the sections of steel plates and the built-up sections for the members of the 3D tied arch. The constraints considered were the design code provisions of AASHTO-LRFD, namely the strength constraints and displacement constraints. In addition, the geometric constraints were considered, to ensure manufacturability of the bridge. It was shown that the proposed enhanced ABC algorithm, yielded lowest overall weight of bridge as compared to the standard ABC algorithm.

Recently, Yousif and Saka (2021) have optimized the design of a steel space frame (600-member building) using an enhanced Beetle Antenna Search (eBAS) algorithm. The design constraints were strength and displacement constraints under the LRFD-AISC design code provisions, as well as the geometric constraints. The design variable was the section of steel members selected from the W-sections standard list. The objective function was the minimization of the total weight of the frame. The

maximum number of iterations needed to obtain the optimum design solution was 50,000 iterations. It was shown that the eBAS had outperformed the standard Beetle Antenna Search Algorithm (BAS) by requiring a smaller number of iterations to yield the optimum design solution. The eBAS also achieved less overall weight as compared to other metaheuristic techniques considered in the study, namely ABC and Differential Evolution (DE).

2.6. Summary

From the above literature review, it can be observed that the design optimization of steel frames has been a research interest for the past few decades. The deterministic methods were the traditional methods used and they have proved to yield optimum design solutions within a minimal number of iterations. However, they require computation of the derivative of the constraint function, which may be complicated in some cases where the functions are highly nonlinear.

One of the most used deterministic methods was the optimality criteria method. It has been noticed in the optimality criteria method, that the design variable was considered as a continuous variable, while in fact the steel profiles must be selected from the standard list of practically available steel sections, which means that the design variable is discrete. Moreover, the optimality criteria method was proved to yield optimum design within a minimal number of iterations, irrespective of the size of the structure.

Later, the metaheuristic methods have evolved and become more widespread. These methods do not require the computation of the constraint function derivative. They are based on random search and evaluation of the objective function, while fulfilling the limitations set by the design constraints. These methods require very large number of

iterations, that could reach up to 50,000 iterations depending on the size of the optimization problem and the design constraints considered.

The above literature review had given the researcher an idea of the various methods used for design optimization of steel space frames. The intent of the researcher is to minimize the number of iterations needed for the design optimization of steel space frames. The researcher intends to benefit from the rapid convergence of the optimality criteria method. This shall be applied for the displacement constraints. As for the strength constraints, the metaheuristics approach shall be adopted. This will be a hybrid optimization algorithm that combines both deterministic and metaheuristic optimization techniques.

Chapter 3

Optimum Design of Steel Space Frames

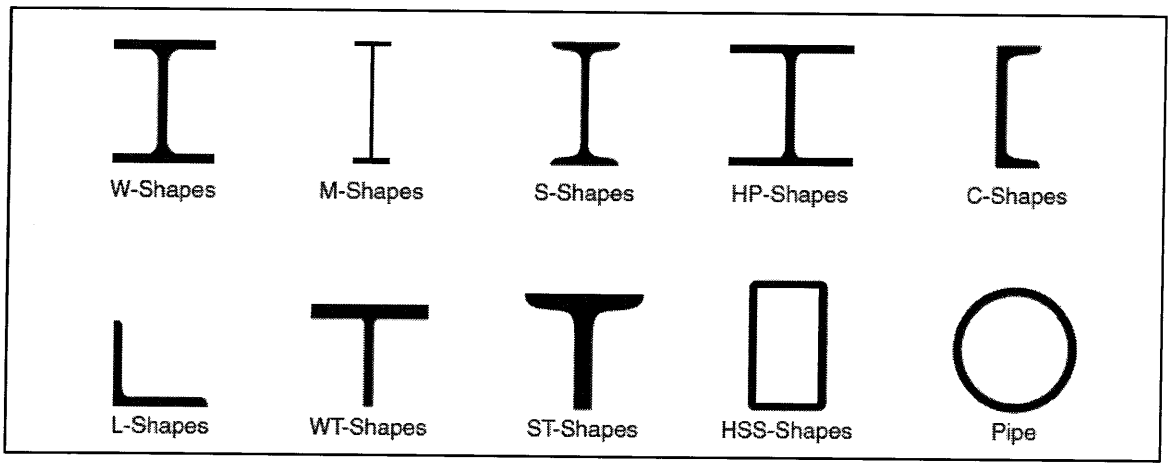


Figure 3.1. Standard rolled shapes (Williams, 2011)

3.3. Design Constraints

The constraints of the steel space frame design optimization considered in this research are the design code provisions set in the LRFD-AISC, namely the strength constraints and the displacements constraints. All members of all groups of the frame shall satisfy these criteria.

3.3.1. Strength Constraints

The Load and Resistance Factor Design (LRFD-AISC) is considered in this research for the design of the space frame members. The steel frame members, both beams and columns, shall be checked for the strength constraint, by considering combined axial force and biaxial bending (beam column design).

3.3.1.1. Combined Axial Force and Bending

According to AISC 360-16 Chapter H, for beam column members that are subject to biaxial bending and axial force (either Tension or Compression) and have doubly or singly symmetric sections, the following interaction formulae need to be satisfied:

(i) When $\frac{P_r}{P_c} \geq 0.2$

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0 \qquad (3.2)$$

(ii) When $\frac{P_r}{P_c} < 0.2$

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0 \qquad (3.3)$$

Where, P_r = required axial strength using LRFD load combinations (N)

M_r = required flexural strength using LRFD load combinations (N.mm)

$P_c = \emptyset_c P_n$ = Design axial strength (N)

$M_c = \emptyset_b M_n$ = Design flexural strength (N.mm)

P_n = Nominal axial strength (N)

M_n = Nominal flexural strength (N.mm)

$\emptyset_b$ = resistance factor for flexure = 0.90

$\emptyset_c$ = resistance factor for compression = 0.90

$\emptyset_t$ = resistance factor for tension

($\emptyset_t$= 0.90 for tensile yielding, $\emptyset_t$= 0.75 for tensile rupture)

x = subscript relating symbol to major axis bending

y = subscript relating symbol to minor axis bending

3.3.1.2. Design Tensile Strength

According to AISC 360-16 Chapter D, and using the LRFD approach, the design tensile strength is calculated as follows:

$$P_c = \emptyset_c P_n = 0.9 F_{cr} A \quad (3.12)$$

The nominal compressive strength is dependent on the slenderness ratio, λ, according to the following:

$\lambda = \frac{KL}{r}$ = slenderness ratio (3.13)

i) When $\lambda \leq 4.71\sqrt{\frac{E}{f_y}}$ (Plastic buckling)

$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) F_y \quad (3.14)$$

ii) When $\lambda > 4.71\sqrt{\frac{E}{F_y}}$ (Elastic buckling)

$$F_{cr} = 0.877 F_e \quad (3.15)$$

Where, P_c = Design compressive strength (MPa)

P_n = Nominal compressive strength (MPa)

$\emptyset_c$ = Resistance factor for compression

F_{cr}= Critical buckling stress (MPa)

K = effective length factor

L = laterally unbraced length of the member (mm)

r = radius of gyration (mm)

A_g = Gross area of the section (mm^2)

$F_e = \frac{\pi^2 E}{\lambda^2}$ = elastic buckling stress (MPa)

E = modulus of elasticity of steel = 200,000 MPa

Figure 3.2 shows the standard column curve and the relation between slenderness ratio and the column critical stress. For smaller values of slenderness ratio, the yielding occurs before reaching the critical buckling

stress. This is known as inelastic mode of failure. While for greater values of slenderness ratio, the critical buckling stress occurs before the yielding. This is known as the elastic mode of failure, and it follows the Euler curve for buckling stress. The point where the critical stress for the elastic mode is equivalent to that of the inelastic mode is known as the transition point.

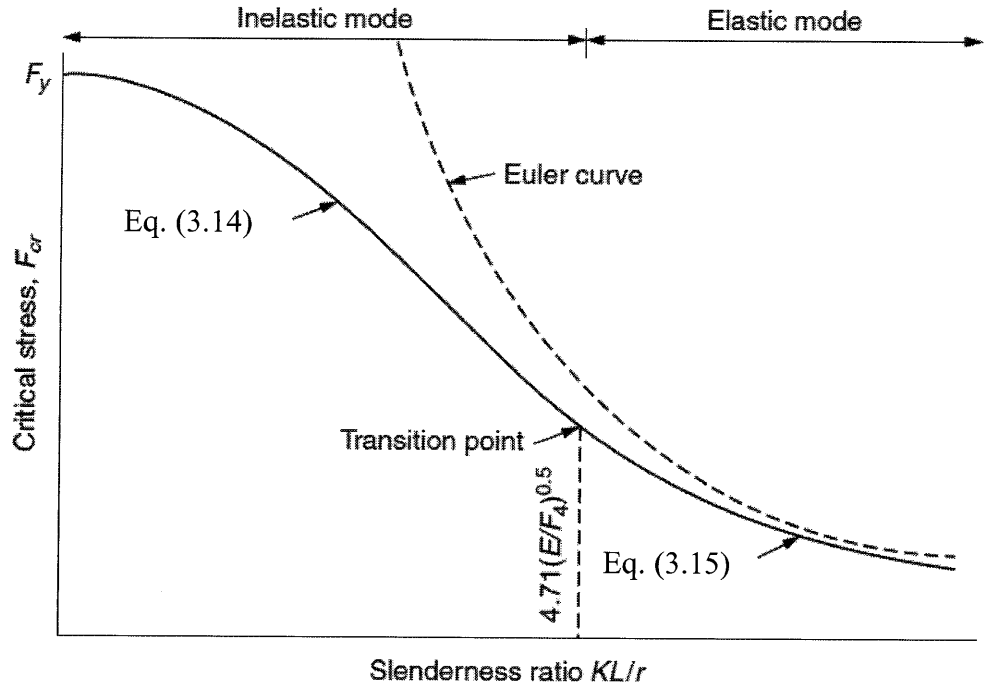


Figure 3.2. Standard column curve (Williams, 2011)

3.3.1.4. Design Flexural Strength about Major Axis

According to AISC 360-16 Chapter F (Section F2) and using the LRFD approach, the design flexural strength for compact I-shaped members bent about their major axis is computed as follows:

$M_{cx} = \emptyset_b M_{nx}$ = Design flexural strength about x-axis (N.mm) (3.16)

$\emptyset_b = 0.90$

For the computation of nominal design flexural strength about major axis, two modes of failure are considered for the flexural member, yielding and

lateral torsional buckling. The nominal flexural strength is taken as the lower of both the calculated strength values.

i) Yielding:

$$M_{nx} = M_{px} = F_y Z_x \qquad (3.17)$$

Where, F_y = minimum yield stress (MPa)

Z_x =plastic section modulus about x-axis (mm^3)

M_{px}= plastic moment about x-axis (N.mm)

ii) Lateral Torsional Buckling:

The nominal flexural strength is dependent on the unbraced length, L_b, which is the length (in mm) between points that are either braced against lateral displacement of compression flange or braced against twist of cross section. The nominal flexural strength is calculated according to the following three cases (Figure 3.3):

(a) When $L_b \leq L_p$, the limit state of lateral torsional buckling does not apply. The nominal strength is calculated according to yielding only, according to equation (3.17).

(b) When $L_p < L_b \leq L_r$

$$M_n = C_b \left[M_p - (M_p - 0.7 F_y S_x) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \leq M_p \qquad (3.18)$$

(c) When $L_b > L_r$

$$M_n = F_{cr} S_x \leq M_p \qquad (3.19)$$

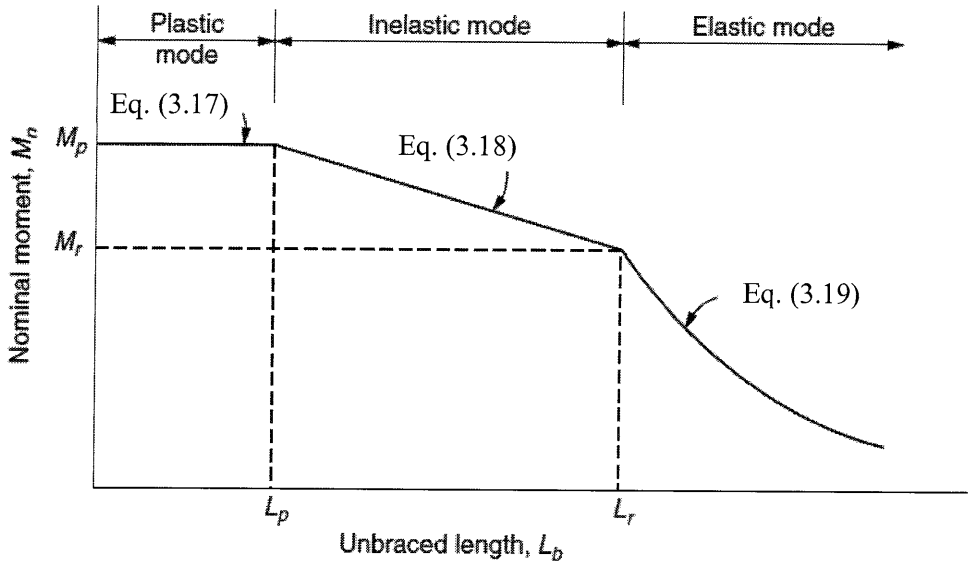


Figure 3.3. Nominal moments and unbraced length (Williams, 2011)

Where, E = modulus of elasticity of steel = 200,000 MPa

S_x= Elastic section modulus taken about the x-axis (mm^3)

F_{cr} = critical stress (MPa)

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2} \sqrt{1 + 0.078 \frac{Jc}{S_x h_0} \left(\frac{L_b}{r_{ts}}\right)^2} \qquad (3.20)$$

L_p = The limiting laterally unbraced length for the limit state of yielding (in mm).

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} \qquad (3.21)$$

r_y= Radius of gyration about y-axis

L_r = The limiting laterally unbraced length for the limit state of inelastic lateral-torsional buckling (in mm).

$$L_r = 1.95 r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_0} + \sqrt{\left(\frac{Jc}{S_x h_0}\right)^2 + 6.76\left(\frac{0.7F_y}{E}\right)^2}} \quad (3.22)$$

$c = 1$ (For doubly symmetric I-shapes)

J = Torsional constant (mm^4)

h_o= Distance between the flange centroids (mm)

$$r_{ts}^2 = \frac{\sqrt{I_y C_w}}{S_x} \quad (3.23)$$

C_w = Warping constant (mm^6)

C_b = Lateral torsional buckling modification factor

$$C_b = \frac{12.5 M_{max}}{2.5 M_{max} + 3M_A + 4M_B + 3M_C} \quad (3.24)$$

M_{max} =Absolute value of maximum moment in the unbraced segment (N.mm)

M_A =Absolute value of moment at quarter point of the unbraced segment (N.mm)

M_B =Absolute value of moment at centerline of the unbraced segment (N.mm)

M_C =Absolute value of moment at three-quarter point of the unbraced segment (N.mm)

The lateral torsional buckling modification factor, C_b, can be conservatively taken as 1. However, to achieve optimum design solution, the actual values shall be computed according to equation (3.24).

3.3.1.5. Design Flexural Strength about Minor Axis

According to AISC 360-16 Chapter F (Section F6) and using the LRFD approach, the design flexural strength for I-shaped members bent about their minor axis is computed as follows:

$M_{cy} = \emptyset_b M_{ny}$ = Design flexural strength about y-axis (N.mm) (3.25)

$\emptyset_b = 0.90$

For the computation of nominal design flexural strength about minor axis, two modes of failure are considered for the flexural member, yielding and flange local buckling. The nominal flexural strength is taken as the lower of both the calculated strength values.

i) Yielding:

$$M_{ny} = M_{py} = F_y Z_y \leq 1.6 F_y S_y \quad (3.26)$$

Where, F_y = minimum yield stress (MPa)

Z_y =plastic section modulus about y-axis (mm^3)

S_y = elastic section modulus about y-axis (mm^3)

M_{py}= plastic moment about y-axis (N.mm)

ii) Flange local buckling: The limit state of flange local buckling does not apply for sections with compact flanges.

According to AISC 360-16, chapter F, all current W shapes, except W150X13, W150X13.5, W200X15, W250X17.9, W150X22.5, W200X46.1, W530X72, W310X97, W360X134, and W360X147 have compact flanges for F_y= 345 MPa. Moreover, all current W shapes have compact webs at $F_y \leq$ 485 MPa. Therefore, these ten sections have been eliminated from the pool of W sections (Appendix A).

3.3.2. Displacement Constraints

Serviceability limit states are the conditions in which the function of a building is affected adversely by excessive deformations. Exceeding the serviceability limitation does not cause the collapse of a building, but it affects the function of a building, causes discomfort of the occupants, and causes damages that are costly to repair.

The displacement constraints in accordance with AISC 360-16, Chapter L, implies that the vertical deflections of members shall not exceed a certain ratio of the span of the member. This is known as the deflection constraint. In addition, the lateral deflection (drift) due to the wind loads, shall not exceed a certain ratio of the height of the building. This is known as the drift constraint.

3.3.2.1. Deflection Constraints

According to Section L2 of AISC 360-16, the allowable vertical deflection of steel frame members under the service loads (unfactored dead and live loads) are limited to 1/360 of the span for floor members and 1/240 of the span for roof members.

3.3.2.2. Drift Constraints

According to Section L3 of AISC 360-16, the accepted range of drift limits due to service wind loads varies from 1/100 to 1/600 times the building height, H. The most widely used value is H/400 for the top storey drift.

The displacement limitations of steel frames according to *cl.* 3.3.2.1 and *cl.* 3.3.2.2. are summarized in Table 3.1.

Table 3.1. Displacement limitations of steel frames.

	Displacement	Limit
1	Floor girder deflection due to service load	L/360
2	Roof girder deflection due to service load	L/240
3	Lateral drift due to service wind load	H/400

3.3.3. Loading and Load Combinations

The design loads considered in the optimization of steel frames design are as follows (Segui, 2007):

1. Dead Loads are the permanent loads including the weight of the structure known as self-weight. They also include the weight of non-structural components such as finishes, partitions, suspended ceiling, mechanical equipment, plumbing fittings and light fixtures.
2. Live Loads are the loads that are not as permanent as dead loads. They are not always acting on the structure at a given time, and their location is varying. These include weight of loose furniture, equipment, and occupants of the building.
3. Wind Loads are caused due to the pressure or suction exerted on the exterior surfaces of a building. Their consideration is more significant for tall structures, especially for lateral displacement limitations.
4. Snow Loads are a type of live loads; however, they are considered as a separate category of loads. They consist of the weight of accumulated snow, especially on the roof of a building.

According to *(ASCE/SEI 7-16)*, the factored load combinations to be considered for the strength design are as follows:

i) 1.2 D + 1.6 L + 0.5 S

ii) 1.2 D + 1.0 L + 1.6 S

iii) 1.2 D + 1.0 L + 0.5 S + 1.0 W

The unfactored load combination considered for deflection constraint is 1.0D + 1.0 L, while the unfactored load combination considered for drift constraint is 1.0 W. Where, D is the dead load, L is the live load, S is the ground snow load and W is the wind load.

3.4. Summary

This chapter has described the optimum design of steel space frames including the objective function, design variables and design constraints. The methods and procedures used for handling these constraints and obtaining the design optimization solutions are described in the next chapter.

Chapter 4

Methods and Procedures

Chapter 4 Methods and Procedures

4.1. Introduction

In Chapter 3, the design optimization problem has been defined in terms of the objective function, design constraints and design variables. The objective function and design constraints have been coded in MATLAB. The design variable is obtained based on the dominant design constraint, and the new value of design variable is used to modify the structural model in SAP2000 and carry out the analysis for the next iteration. The response of the structure is then fed to the MATLAB routine, until the convergence is obtained. The integration of the MATLAB code and SAP2000 is achieved by using Open Application Programming Interface (OAPI).

The methods and procedures used to handle each of the design constraints and carry out the design optimization are described in this chapter. In other words, this chapter describes the steel space frame design optimization algorithm, that was developed for this research.

4.2. SAP2000 Modelling

SAP2000 software is a structural analysis and design software developed by Computers and Structures Incorporation (CSI). It is a finite element based structural analysis and design for Civil Engineering structures. It has sophisticated analysis techniques for complex projects. (CSI, 2011).

4.2.1. Steel Space Frame Model

In this study, a steel space frame that was previously optimized by using Metaheuristic algorithms (Carbas, 2016) has been selected and modelled in SAP2000. The selected frame consists of five stories and two bays. 105 members of this regular steel space frame are connected to each other rigidly. Each bay is 6 m wide, and each storey is 4 m high. Figure 4.1.

Figure 4.2. shows the 105-member steel space frame, that is modelled in SAP2000.

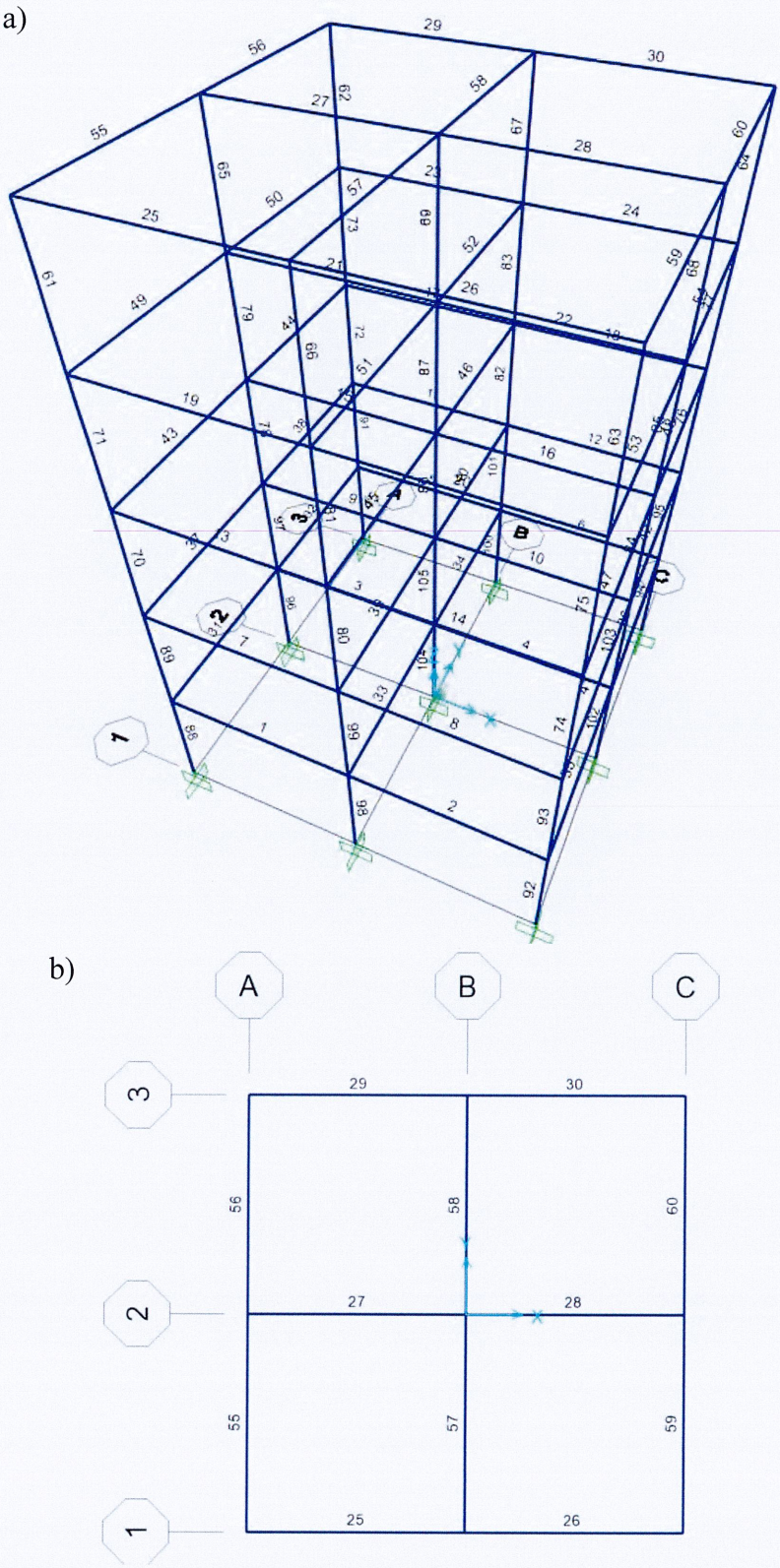


Figure 4.2. 105-member steel space frame SAP2000 model: (a) 3D view, (b) Plan view.

The steel space frame consists of 105 members that are divided into 11 groups in SAP 2000 model according to Table 4.1.

Table 4.1. Steel space frame member groups in SAP2000 model

Group	Member Labels in SAP2000	Number of Members in Group
1	1 – 30	30
2	31 – 60	30
3	61 – 64	4
4	65 – 68	4
5	69	1
6	70 – 77	8
7	78 – 85	8
8	86 – 87	2
9	88 – 95	8
10	96 – 103	8
11	104 – 105	2
	Total Number of Members	105

4.2.2. Steel Space Frame Model Loading

The unfactored uniformly distributed loads (UDL) assigned on beams as per (Carbas, 2016) are summarized in Table 4.2. Moreover, the basic wind speed is 65 mph (105 kph).

Table 4.2. Beam gravity loading of the 105-member regular steel space frame (Carbas, 2016)

	Uniformly Distributed Load (kN/m)		
Beam Type	**Dead Load**	**Live Load**	**Snow Load**
Roof Beams	4.78	-	1.508
Floor Beams	4.78	5.76	-

The load combinations considered are as follows (Carbas, 2016):

i) 1.2D + 1.6L + 0.5S

ii) 1.2D + 0.5L + 1.5S

iii) 1.2D + 0.5L + 0.5S + 1.6W

Where, D is dead load, L is live load, W is wind load and S is snow load.

Figure 4.3. shows the dead load applied on beams of the 105-member steel space frame. Similarly live load and snow load have been applied in accordance with Table 4.2.

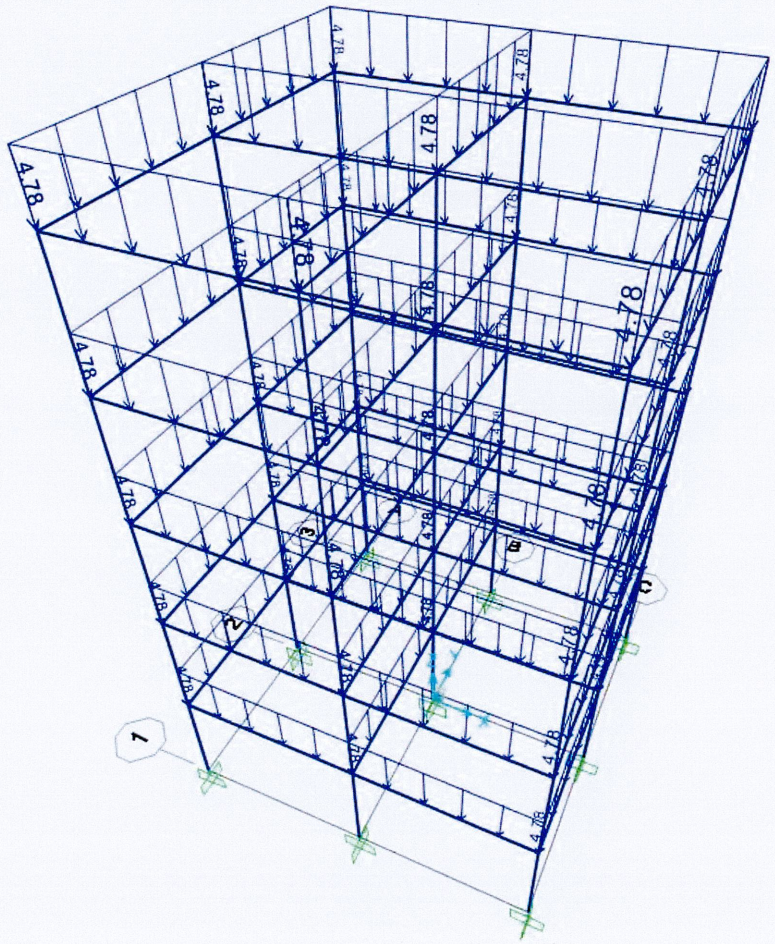


Figure 4.3. Dead load (kN/m) assigned on beams of the steel space frame

For the wind load, the SAP2000 feature of Auto wind load calculation is used in accordance with ASCE 7-16. The equivalent joint point loads for the centrically assigned wind load in each of the X direction and Y direction are shown in Figure 4.4 and Figure 4.5, respectively. In order ensure that the wind load is calculated properly, a rigid diaphragm is assigned to the joints of each level of the space frame.

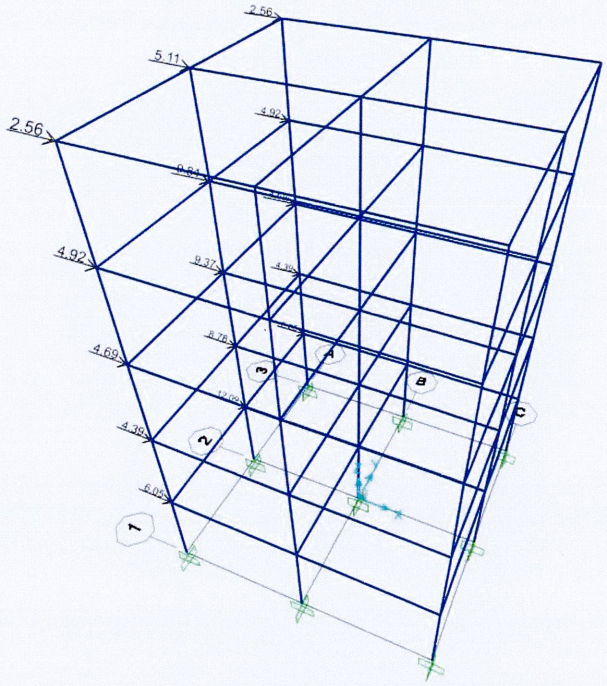


Figure 4.4. Wind load in X direction (Auto SAP2000 assigned)

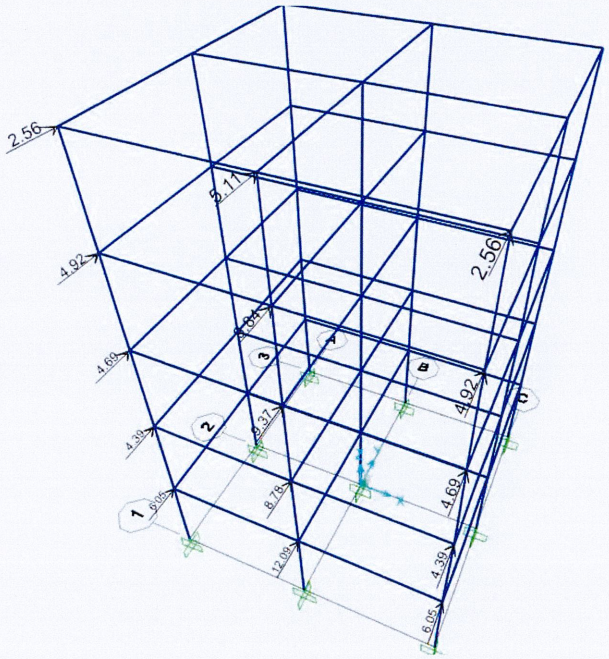


Figure 4.5. Wind load in Y direction (Auto SAP2000 assigned)

4.2.3. SAP2000 Analysis Results

Structural analysis is carried out by SAP2000. The results from the first order analysis generates the internal member forces, namely V2, V3, P, M2, M3 and T, where V2 is the shear force in minor axis, V3 is the shear force in major axis, P is the axial force, M2 is the bending moment about minor axis of the member, M3 is the bending moment about major axis of the member and T is the torsional moment. Figure 4.6. shows the bending moment diagrams (M3) under the factored load combination (1.2D + 1.6L + 0.5S), named as ULS-1 in the SAP2000 model.

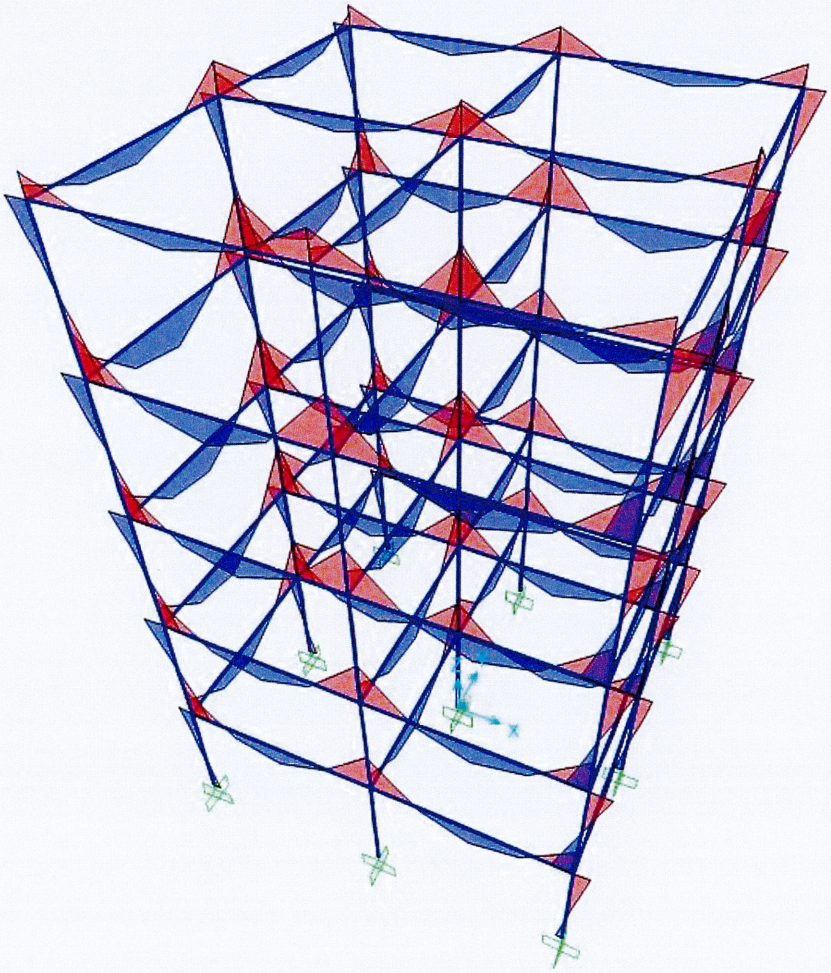


Figure 4.6. Diagram of bending moment about major axis under ULS-1 load combination

The analysis results of a typical frame member are shown in Figure 4.7. All members of the frame have been assigned 5 output stations, in other words the structural analysis of SAP2000 generates the member forces at 5 locations along the span of the member as follows:

i) First end of the member.
ii) One quarter of the span of the member.
iii) Midspan of the member.
iv) Three quarter of the span of the member.
v) Second end of the member.

These values are needed to calculate the strength constraint parameters described in Chapter 3.

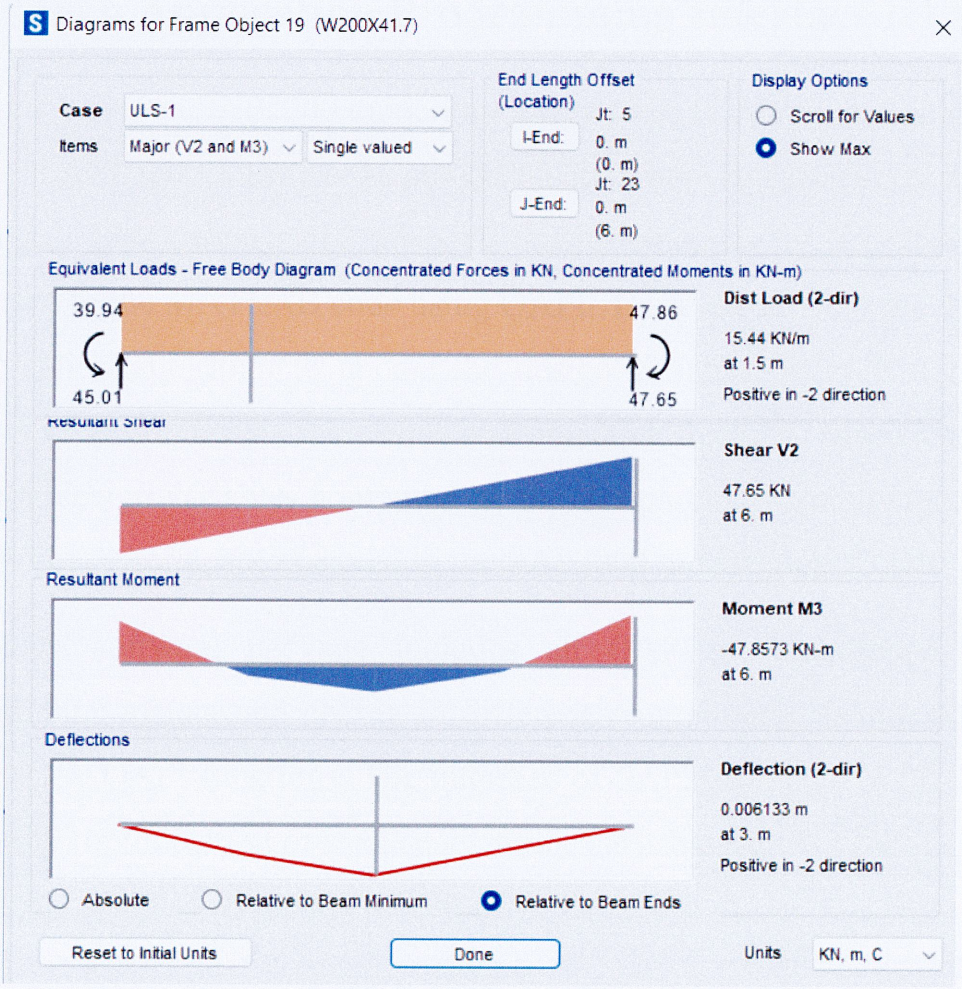


Figure 4.7. Analysis results of a typical frame member

Table 4.3. OAPI functions used for Design Optimization of Steel Space Frame

	OAPI Function	Objective
1	SapModel.InitializeNewModel	Open SAP2000 software
2	File.OpenFile	Open an existing model
3	SapModel.SetPresentUnits	Change the units in SAP model
4	Analyze.RunAnalysis	Run analysis of model
5	AnalysisResultsSetup.Deselect-AllCaseAndComposForOutput	Deselect all load combinations before generating the results.
6	AnalysisResultsSetup.Set-CompoSelectedForOutput	Select a particular load combination for analysis results.
7	SapModel.Results	Get analysis results
8	AnalysisResults.FrameForce	Get member internal forces
9	AnalysisResults.JointDispl	Get joint displacements
10	SapModel.SetModelIsLocked	Unlock the model after analysis
11	FrameObj.SetSection	Set new section to a frame member or group of members
12	File.Save	Save the model
13	SapObject.ApplicationExit	Close the SAP2000 file

The above functions have been coded in MATLAB to achieve the interface between MATLAB code and the SAP2000 existing model, that was previously modelled separately. Some of the applications of OAPI are presented in Appendix B. Moreover, the results obtained from SAP2000 by OAPI were stored in a set of arrays that were initialized in MATLAB.

In conclusion, OAPI has proved to be a very useful feature of SAP2000, that facilitated the design optimization process of steel space frames.

4.4. Optimality Criteria Method

The displacement constraints described in *cl.* 3.3.2 are handled using the optimality criteria method. Optimality criteria method is used to compute the lower bound of the design variable, based on the displacement constraints. The method is independent of the size of the structure. It uses the lagrangian multipliers to convert the constrained problem into an unconstrained one (Saka & Geem, 2013). The method also considers continuous design variables, however the design variables in this case are discrete, as the sections of steel profiles must be selected from a set of standard steel sections, namely the American W-sections.

4.4.1. Mathematical Formulation

In case of dominance of displacement constraints, the lower bounds of the design variables are determined by the optimality criteria method. The mathematical model for this method is obtained by considering the displacement constraints only. The method is based on the virtual work method and to obtain the results, the steel space frame is modelled under two load cases. In case of drift constraint, the model is analyzed under two load cases: (i) the unfactored wind load (ii) unit point load in the direction of restricted displacement. The displacement vectors of all joints of the frame under of each of the two load combinations are obtained by SAP2000 analysis.

The computation of the design variables' lower bound values for each group of members based on the optimality criteria method can be summarized as follows (Saka & Geem, 2013):

$$A_k^{v+1} = A_k^v \left[\frac{1}{\rho \sum_{i=1}^{mk} l_i} \sum_{l=1}^{nlc} \sum_{J=1}^{P} \frac{\lambda_{il}}{{A_k}^2} \sum_{j=1}^{mk} f_{ijl} \right]^{1/r} \quad ,k = 1, \dots, ng \qquad (4.1)$$

Where, A_k is the cross-sectional area of members belonging to group k.

ν is the current iteration number.

$\nu + 1$ is the next iteration number.

ng is the total number of member groups.

mk is the total number of members in group k.

ρ = Density of steel = 76.97 kN/m^3

l_i = length of member in the group k

nlc is the total number of load cases that the frame is subjected to.

p is the total number of restricted displacements.

λ_{jl} is the LaGrange parameter (equation 4.2).

f_{ijl} is the flexibility coefficient of a member (equation 4.3).

r is a preselected constant known as step size.

The value of the LaGrange parameter, λ, is changed in every iteration according to the following:

$$\lambda_{jl}^{\nu+1} = \lambda_{jl}^{\nu}\left(\frac{\delta_{jl}}{\delta_{jlu}}\right)^{1/c} \quad , \quad j = 1, \ldots, p \tag{4.2}$$

Where, δ_{jl} is the displacement of joint j under the load case l

δ_{jlu} is the upper bound of the displacement of joint j

c is a preselected constant known as a step size

$$f_{ijl} = X_{il}^T K_i X_{ij} A_i \tag{4.3}$$

Where, X_{il} is the displacement vector of member i due to load case l.

X_{ij} is the displacement vector of member i due to unit point load in the direction of restricted displacement at joint j.

K_i is the contribution of member i to the overall stiffness matrix.

The above equations have been coded in MATLAB. Accordingly, the lower bounds of the design variables for each group of members in each iteration of the optimization process are computed based on the displacement constraints. Section 4.5 describes the method by which the lower bound of design variables for each group of members are determined based on strength constraints. In each iteration, the larger of these two values is used to select the W-section from the list of practically available sections for each group of members.

4.5. Beetle Antenna Search Algorithm

The strength constraints described in *cl.* 3.3.1 are handled using the beetle antenna search algorithm. Beetle antenna search algorithm is a metaheuristic algorithm inspired by the searching behavior of long-horn beetles (Jiang & Li, 2018). The algorithm can be summarized in five stages; initialization of beetle position, randomizing the direction of movement, estimating right-hand-side and left-hand-side movement, moving in best direction, and updating the sensing length and step size. The steps of this algorithm are described as follows (Jiang & Li, 2018; Yousif & Saka, 2021):

i) Initialization of Beetle position: Initialize the beetle position x, where $x = \{x_1, x_2, \ldots, x_n\}$, n is the total number of variables. Compute the objective function $f(x)$ for this initial position. Initialize parameters d^0 and δ^0. The beetle initial position is estimated using the equation below:

$$x_i = x_{li} + rand(0,1) \times (x_{ui} - x_{li}) \quad (4.4)$$

Where, x_{li} and x_{ui} are the lower and upper bounds of variable i, respectively.

ii) Randomization of movement direction: the movement direction b is randomized for each of the variables. The movement direction value is determined using the equation below:

$$b_i = \frac{rand\ s_i}{||rand\ s(n,1)||} \tag{4.5}$$

Where, rands$_i$ is a random number ranging from -1 to 1 for variable i.

$||rand\ s(n,1)||$ is the norm of the randomized values of all the variables.

iii) Estimation of right-hand-side and left-hand-side movement: the right-hand-side and left-hand-side movements, $x_{(right)}$ and $x_{(left)}$ are estimated for a given sensing antenna length (d). They are used to determine the objective value and the best movement direction. They are determined using the equation below:

$$x_{i(right)} = x_i + d_t \times b_i \tag{4.6}$$

$$x_{i(left)} = x_i - d_t \times b_i \tag{4.7}$$

Where, d^t is the sensing length of antenna at t^{th} iteration.

iv) Movement in the best direction: the best fit of both values computed in equations (4.6) and (4.7) is used as a direction for beetle movement. Based on the step size δ, the beetle moves in the better direction, as shown in equation (4.8). Moreover, the value of the objective function is evaluated, to compare against the current solution.

$$x_i^{t+1} = x_i^t - \delta^t \times b_i \times sign\left(f(x_{(right)}) - f(x_{(left)})\right) \tag{4.8}$$

Where, δ^t is the step size at t^{th} iteration.

v) Update of sensing length and step size: the sensing length d and the step size δ are updated by the following equations:

$$d^{t+1} = 0.95d^{t} + 0.01 \tag{4.9}$$

$$\delta^{t+1} = 0.95\delta^{t} \tag{4.10}$$

Where, δ^{t+1} is the step size for the next iteration.

d^{t+1} is the sensing length for the next iteration.

These steps are repeated until the termination criterion is satisfied. The algorithm parameters are the number of beetles (population size) and the maximum number of objective function evaluations. The MATLAB code for the BAS algorithm was taken from the literature (Jiang & Li, 2018).

To evaluate the objective function in BAS algorithm, the following equation was used (Carbas, 2016):

$$\mathrm{W_p} = W(1 + C)^{\varepsilon} \tag{4.11}$$

Where, W_p is the penalized weight of the structure.

C is the value of the total constraint violations.

ε is a preselected constant known as penalty coefficient.

For the strength constraints and based on equations (3.2) and (3.3) of Chapter 3, the constraints violations can be evaluated as follows:

(i) When $\frac{P_r}{P_c} \geq 0.2$

$$g_i = \frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0 \tag{4.12}$$

(ii) When $\frac{P_r}{P_c} < 0.2$

$$g_i = \frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0 \tag{4.13}$$

Then, the total constraints violations, C, is calculated as follows:

$$C = \sum_{i=1}^{nc} c_i \tag{4.14}$$

$$c_i = 0 \text{ , if } g_j \leq 1 \tag{4.15}$$

$$c_i = g_j \text{ , if } g_j > 1 \tag{4.16}$$

Where, g_j is the j^{th} constraint function.

nc is the total number of constraints.

It should be noted that based on the strength constraint, the design variable is computed for each member of the group, to determine the member that is subjected to the most critical combined forces. Then, the maximum value of design variable for all members of a particular group, is selected for all members of that group.

4.6. Combined Optimization Algorithm

The lower bound of design variables values computed based on the displacement constraints (Section 4.4) was compared with the lower bound of design variables values computed based on the strength constraint (Section 4.5). The maximum of these two values was used to select the next biggest section in the W-section list for each group of members in each iteration of the optimization process.

The algorithm developed in this research for the optimization of steel space frames combines both deterministic methods and metaheuristic methods to achieve the optimum design solution. A summary of the design optimization algorithm used in this research is presented as a flow chart in Figure 4.8.

The algorithm parameters that need to be determined are summarized in Table 4.4.

Table 4.4. Parameters of Steel Space Frame Design Optimization Algorithm

	Constraint	Parameter
1	Strength	Number of beetles (population size).
2		Number of objective function evaluations.
3		Penalty coefficient, ε. (Equation 4.11).
4	Displacement	Initial value of λ. (Equation 4.2).
5		Step size, c. (Equation 4.2).
6		Step size, r. (Equation 4.1).

This chapter has described the adopted methods and the procedures used to obtain the optimum design solution for steel space frames. The obtained results and discussion of these results are presented in the next chapter.

Chapter 5

Results and Discussion

Chapter 5 Results and Discussion

5.1. Introduction

In chapter 4, the detailed methods and procedures adopted for the optimization of steel space frames have been described. The algorithm developed is a combination of both deterministic and metaheuristic methods. In this chapter, the obtained results of the design optimization of steel space frames are presented and discussed. Initially, the design of a single storey single bay steel space frame is optimized to demonstrate the efficiency of the developed algorithm and to analyze the obtained results in a simplified way. Later, the five-storey steel space frame presented in section 4.2 is optimized and the optimum design solution is presented and discussed.

5.2. Single Storey Single Bay Steel Space Frame

A single storey single bay steel space frame is selected for simplicity, to ensure that the method adopted is effective and that convergence is obtained in a minimum number of iterations (structural analysis).

The selected frame has a storey height of 3 m and a bay width of 3 m. The selected steel grade is A992 of the American Society of Testing and Materials (ASTM), with yield strength with minimum yield stress, $F_y = 50$ ksi (345 MPa). The beams are subjected to a dead load of 30 kN/m and live load of 35 kN/m. The wind load (lateral load) is assumed to be two point loads of 50 kN each, acting in the positive x direction at joints 1 and 2, respectively. Figure 5.1. shows the single storey steel space frame as modelled in SAP2000. The frame members have been divided into two groups, where one includes the beam members of the frame and the other includes the column members of the frame, respectively.

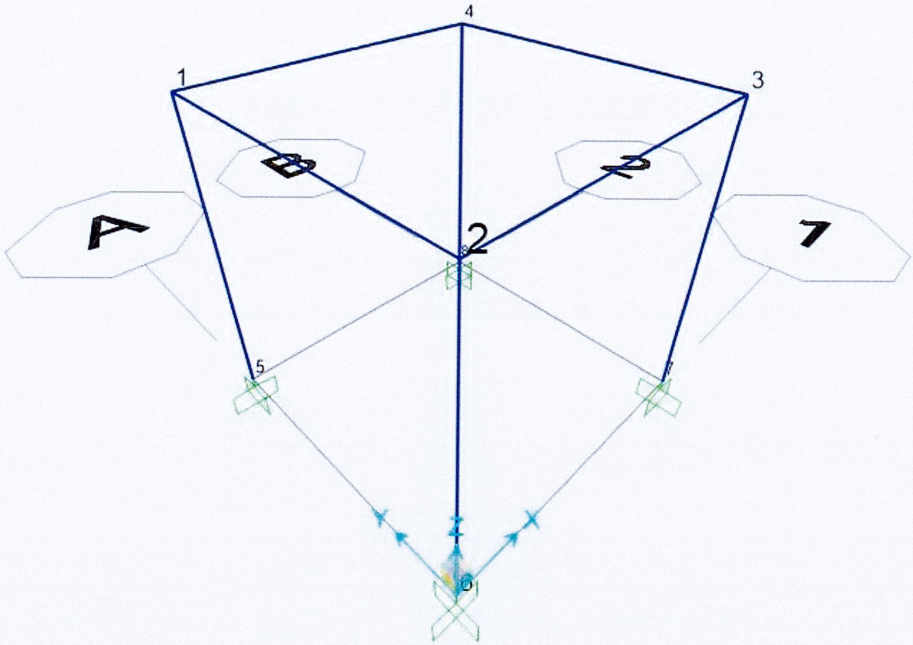


Figure 5.1. Single Storey Single Bay Steel Space Frame

The dead load assigned to the beam members of the frame, in addition to its self-weight, is shown in Figure 5.2.

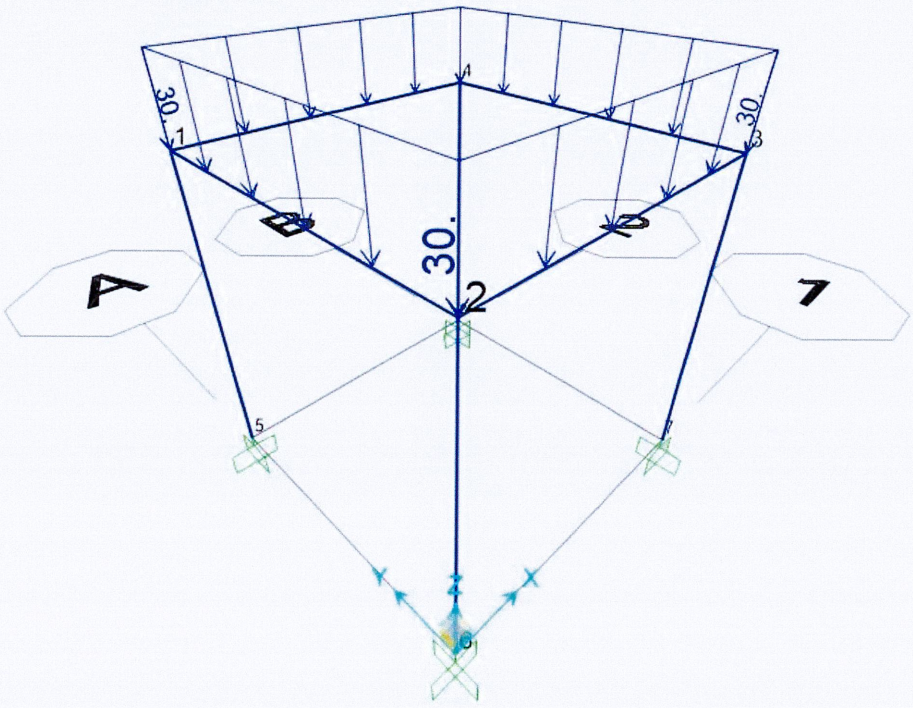


Figure 5.2. Dead Load (kN/m) Assigned to Beam Members of Single Storey Frame

The live load assigned to the beam members of the frame is shown in Figure 5.3.

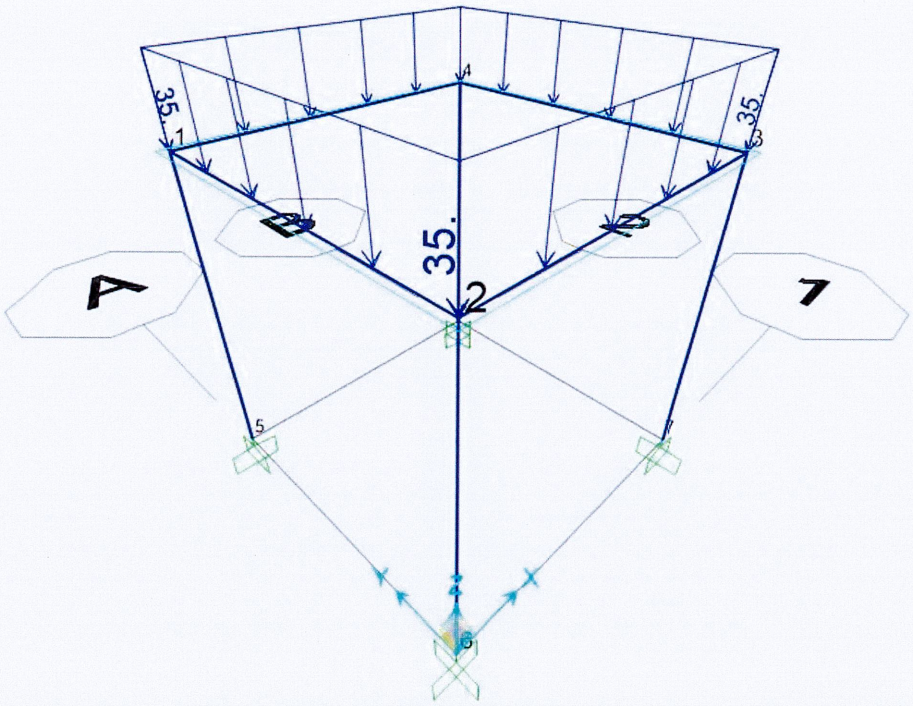


Figure 5.3. Live Load (kN/m) Assigned to Beam Members of Single Storey Frame

The assumed wind load assigned to joints 1 and 2 of the frame is shown in Figure 5.4.

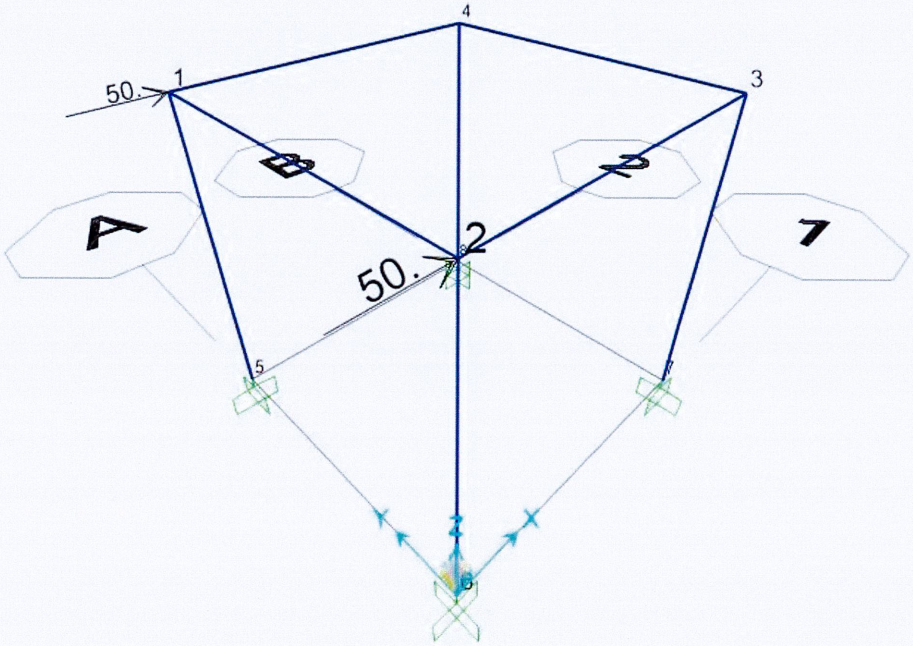


Figure 5.4. Wind Load (kN) Assigned to The Single Storey Frame Joints

5.2.1. Optimization for Displacements Constraints Only

To demonstrate the effectiveness of the optimality criteria method, the single storey steel space frame is optimized taking into consideration the drift constraints only. This also allows for the identification of the algorithm parameters related to the drift constraints, independently.

To carry out the optimality criteria method (described in section 4.4), the model is analyzed under two load cases, the first is the wind load (shown in Figure 5.4), and the second is a unit load in the positive x direction at joint 1, which is the displacement that needs to be restricted. The top storey drift is limited to 7.5 mm (H/400). The initial section assigned to both beam members and column members is W200X31.3. The results of the optimization for drift constraints only, are shown in Figure 5.5. These results are obtained after several trials with different parameters.

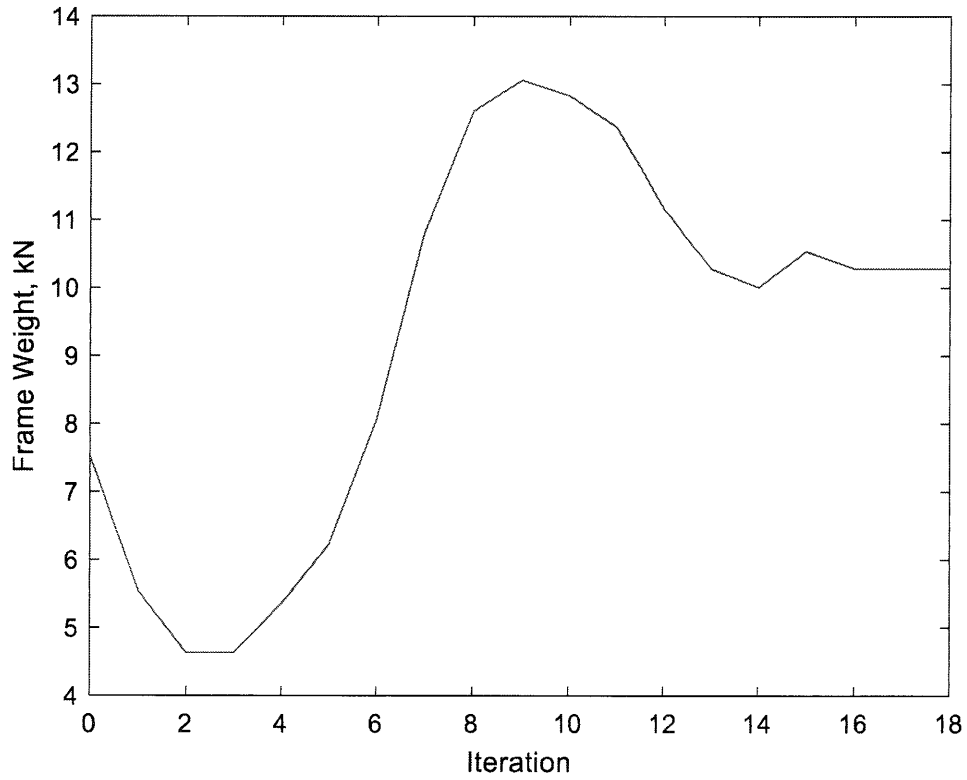


Figure 5.5. Single Storey Frame Optimization According to Drift Constraints

After several trials and by changing the algorithm parameters related to the drift constraints in each trial, the convergence of the structure weight is obtained, and the drift constraints are satisfied. The optimum design sections for beam members and column members are W200X41.7 and W360X44, respectively. The minimum frame weight obtained is 10.28 kN and the maximum top storey drift is 6.42 mm, which is very close to its upper bound of 7.5 mm. Moreover, the number of iterations needed to achieve the optimization results is less than 20 iterations, which is a very small number of iterations (structural analysis). The design optimization results according to drift constraints are summarized in Table 5.1, while the best values for the algorithm parameters are presented in Table 5.2.

Table 5.1. Results of Single Storey Frame Design Optimization to Drift Constraints

	Description	Result
1	Optimum design section of beams group.	W200X41.7
2	Optimum design section of columns group.	W360X44
3	Minimum frame weight (kN)	10.28
4	Maximum top storey drift (mm)	6.42
5	Number of iterations	18

Table 5.2. Algorithm Parameters of Drift Constraints for Single Storey Frame

	Parameter	Value
1	Initial value of λ. (Equation 4.2).	100
2	Step size, c. (Equation 4.2).	1
3	Step size, r. (Equation 4.1).	10

The above results indicate that the adopted optimality criteria method is efficient in handling the drift constraints of the design optimization algorithm and yielding the results within a minimum number of iterations.

5.2.2. Optimization for Strength Constraints Only

To verify the efficiency of the Beetle Antenna Search Algorithm (described in section 4.5) in handling the strength constraints of the optimization algorithm, the design optimization of the single storey steel space frame is carried out by considering the strength constraints only. This allows the identification of the algorithm parameters related to the strength constraints, independently.

The initial section assigned to both beam members and column members is W200X31.3. The results of the design optimization for strength constraints only are shown in Figure 5.6.

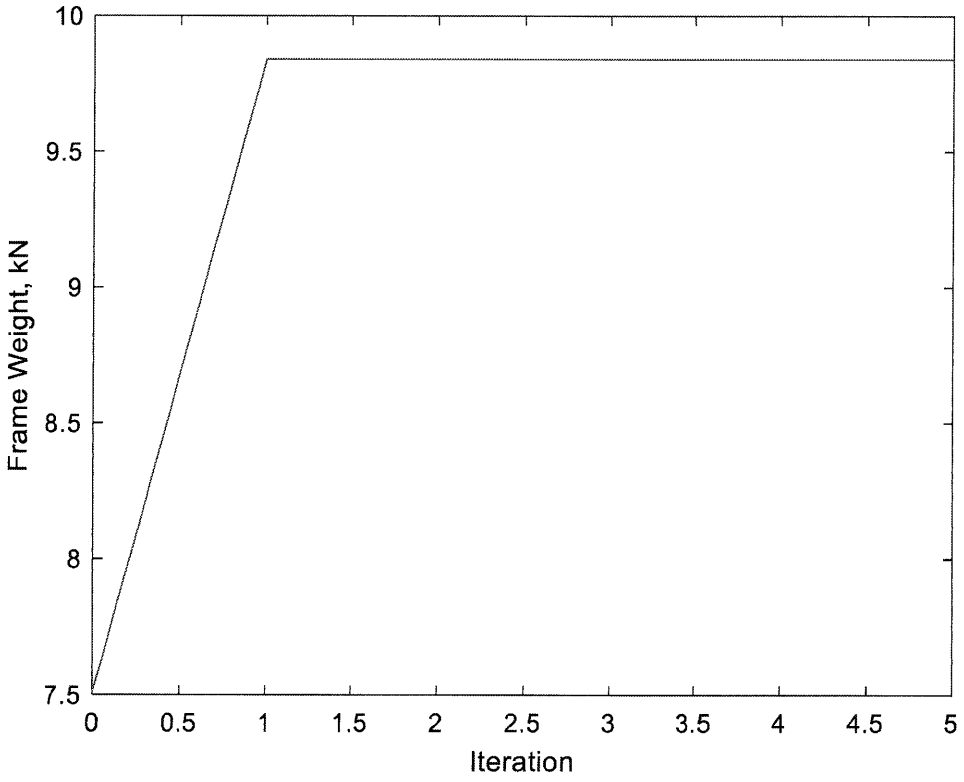


Figure 5.6. Single Storey Frame Optimization According to Strength Constraints

The optimum design sections for beam members and column members, according to strength constraints, are W200X35.9 and W410X46.1, respectively. The minimum frame weight obtained is 9.84 kN and the maximum strength constraint ratio is 0.918, which is very close to its upper bound of 1.0. Moreover, the number of iterations needed to achieve the optimization results is less than 5 iterations, which is a very small number of iterations (structural analysis). The design optimization results according to strength constraints are summarized in Table 5.3, while the best values for the algorithm parameters are presented in Table 5.4.

Table 5.3. Results of Single Storey Frame Optimization to Strength Constraints

	Description	Result
1	Optimum design section of beams group.	W200X35.9
2	Optimum design section of columns group.	W410X46.1
3	Minimum frame weight (kN)	9.84
4	Maximum strength constraint ratio	0.918
5	Number of iterations	5

Table 5.4. Algorithm Parameters of Strength Constraints for Single Storey Frame

	Parameter	Value
1	Number of beetles (population size).	5
2	Number of objective function evaluations.	5
3	Penalty coefficient, ε. (Equation 4.11).	1.5

The above results show that the beetle antenna search algorithm is efficient in handling the strength constraints and yielding the optimum design results in a minimum number of iterations.

5.2.3. Overall Design Optimization of Single Storey Frame

The overall design optimization algorithm (described in section 4.6) was applied to the single storey single bay steel space frame, using the parameters obtained earlier (Table 5.2 and Table 5.4). For the frame weight variation, the same graph obtained for optimization with strength constraints only, was obtained (Figure 5.7). This indicated that the strength constraints are dominant over the displacement constraints, in this case. The maximum strength ratio (combined axial and bending) is 0.912. The maximum shear stress ratio is 0.47, which shows that the shear stress ratio is not a critical constraint for the steel space frame. Moreover, the maximum top storey drift is 5.7 mm, which is well within the allowable limit of 7.5 mm. It should be noted that, when the frame was optimized for drift constraint only, the maximum top storey drift was 6.4 mm. However, when the strength constraint was taken into consideration and due to its dominance, the top storey drift was further reduced.

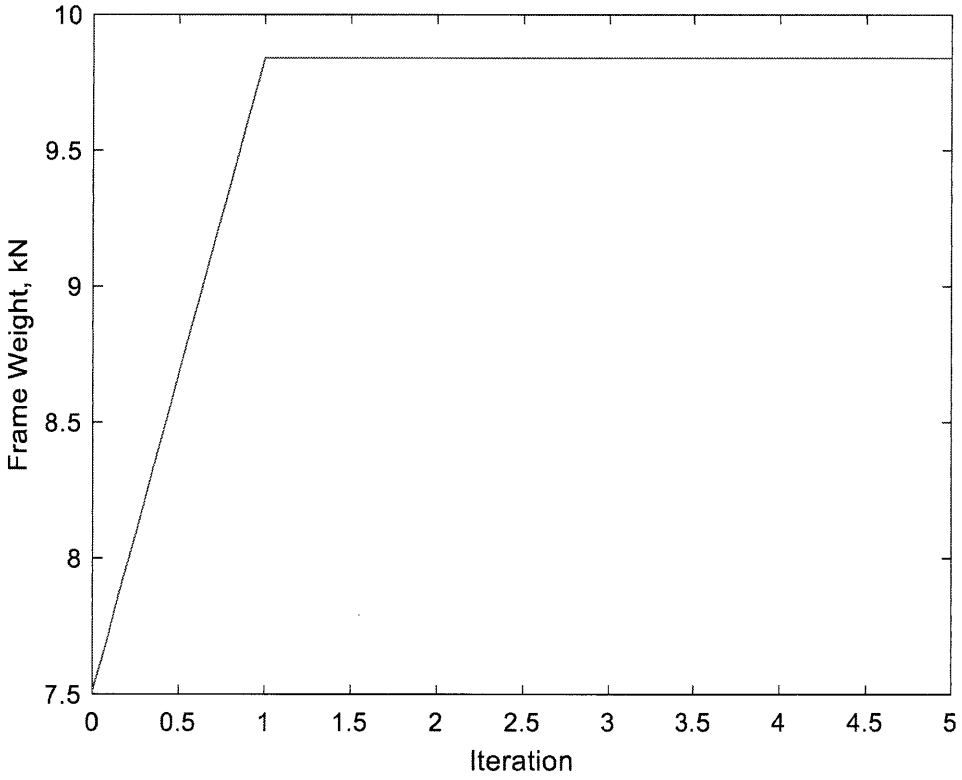


Figure 5.7. Single Storey Frame Overall Design Optimization

The results obtained for the single storey single bay steel space frame are summarized in Table 5.5 and the obtained design optimization algorithm parameters are summarized in Table 5.6.

Table 5.5. Results of Single Storey Frame Design Optimization

	Description	Result
1	Optimum design section of beams group.	W200X35.9
2	Optimum design section of columns group.	W410X46.1
3	Minimum frame weight (kN)	9.84
4	Maximum strength constraint ratio	0.918
5	Maximum top storey drift (mm)	5.7
6	Number of iterations	5

Table 5.6. Design Optimization Algorithm Parameters for Single Storey Frame

	Constraint	Parameter	Value
1	Strength	Number of beetles (population size).	5
2		Number of objective function evaluations.	5
3		Penalty coefficient, ε. (Equation 4.11).	1.5
4	Displacement	Initial value of λ. (Equation 4.2).	100
5		Step size, c. (Equation 4.2).	1
6		Step size, r. (Equation 4.1).	10

The above results show that the combined algorithm developed for the design optimization of steel space frames is efficient and reliable in handling the design constraints and yielding the optimum design solution within a small number of iterations (structural analysis). This indicates that this algorithm can achieve rapid design optimization of steel space frames under LRFD-AISC design code provisions.

5.3. Five Storey Steel Space Frame

The design optimization algorithm is applied to the five-storey steel space frame that was described in Section 4.2. The initial section used for all 11 groups was W200X41.7. The results obtained are summarized in Table 5.7.

Table 5.7. Results of Five Storey Frame Design Optimization

	Description	Result
1	Optimum design section of group 1.	W250X28.4
2	Optimum design section of group 2.	W310X21
3	Optimum design section of group 3.	W760X350
4	Optimum design section of group 4.	W760X582
5	Optimum design section of group 5.	W310X21
6	Optimum design section of group 6.	W610X92
7	Optimum design section of group 7.	W360X44
8	Optimum design section of group 8.	W530X85
9	Optimum design section of group 9.	W530X150
10	Optimum design section of group 10.	W310X117
11	Optimum design section of group 11.	W360X51
12	Minimum frame weight (kN)	378.72
13	Maximum strength constraint ratio	0.942
14	Maximum top storey drift (mm)	49.0
15	Number of iterations	58

The design optimization algorithm parameters used are summarized in Table 5.8.

Table 5.8. Design Optimization Algorithm Parameters for Five Storey Frame

	Constraint	Parameter	Value
1	Strength	Number of beetles (population size).	5
2		Number of objective function evaluations.	5
3		Penalty coefficient, ε. (Equation 4.11).	2.0
4	Displacement	Initial value of λ. (Equation 4.2).	10
5		Step size, c. (Equation 4.2).	2
6		Step size, r. (Equation 4.1).	5

The variation of frame weight against the number of iterations is shown in Figure 5.8.

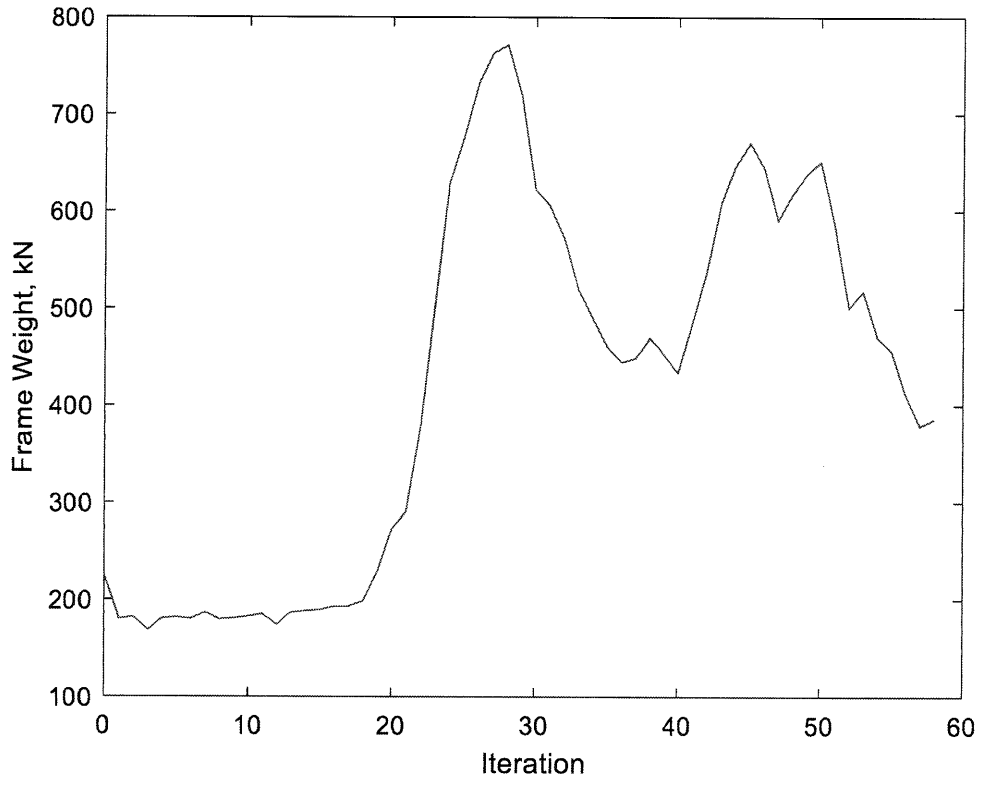


Figure 5.8. Five Storey Frame Overall Design Optimization

The minimum weight achieved by the optimization algorithm was 378.72 kN. This is 3.16% lower than the weight achieved by the FireFly algorithm (FFA), (Carbas, 2016). However, the achieved weight of 378.72 kN is 33.33% higher than that achieved by the Enhanced FireFly algorithm (EFFA), (Carbas, 2016).

Nevertheless, the number of iterations needed to reach to the optimum design solution is drastically reduced. The number of iterations did not exceed 60 iterations, as compared to 50,000 iterations (Carbas, 2016).

In addition, the maximum top storey drift is 49.0 mm, which is very close to its upper bound of 50.0 mm (H/400).

The above shows that the optimality criteria method, when paired with the metaheuristic method, reduced the number of iterations drastically. However, further study is needed to further minimize the frame weight and yield a more economical design.

It should be noted that the upper storey columns achieved by the optimization algorithm, are usually larger than the lower storey columns. This is mainly due to the reason that the geometric constraints were not considered in this study. This can be tackled by further studies to take the geometric constraints into account. Alternatively, the columns can be regrouped in such a way that a column section does not change from storey to the other. In other words, the frame members can be divided into five groups instead of eleven groups. However, the latter approach would not lead to an economical design.

Chapter 6

Conclusion and Recommendations

Chapter 6 Conclusion and Recommendations

6.1. Research Findings and Conclusion

The objectives of the research were summarized in Chapter 1, and the findings related to each of the objectives are summarized in this chapter.

Objective 1

"To develop a structural optimization algorithm for three-dimensional (3D) steel frames with rigid beam column connections."

In this research, a design optimization algorithm for rigidly connected steel space frames was developed. The optimization algorithm made use of both deterministic and metaheuristic approaches. The formulation of design constraints, the optimality criteria method and the BAS algorithm were all coded in MATLAB. The frame was modelled in SAP2000. The interface between MATLAB and SAP2000 was achieved using the OAPI commands, which proved to be useful and efficient in the design optimization procedure.

The algorithm used has yielded optimum design solutions in a minimal number of iterations (less than 60 iterations for a five-storey steel space frame). The weight of the frame was minimized, while satisfying the strength constraints and displacements constraints in accordance with LRFD-AISC.

However, further development is required, such as including the geometric constraints in the design optimization procedure. Moreover, further study is needed to further minimize the frame weight and yield more economical designs.

Objective 2

"Determine the most economical steel profiles for the members of the frame, so that the response of the frame under the external loads satisfies the design code provisions (LRFD-AISC)."

In this research, the frame members were divided into groups. The steel profiles were selected from the list of available W-sections. The frame weight was minimized, while satisfying the strength constraints and displacement constraints in accordance with LRFD-AISC.

Objective 3

"Minimize the number of iterations needed to reach to the optimum design solution."

In this research, the use of optimality criteria method for handling the displacement constraints, had contributed to drastic reduction in the number of iterations (structural analysis) needed to achieve the optimum design solution. The optimality criteria method is a deterministic method, which is used in the algorithm to find the optimum design solution more rapidly.

Further study is needed to develop a deterministic approach for the strength constraints, to reduce the number of iterations needed to achieve the optimum design solution.

6.2. Recommendations for Further Study

1. To develop a deterministic approach for the strength constraint and combine it with the deterministic approach developed for displacement constraint.
2. To combine the optimality criteria approach with other stochastic solution techniques, other than Beetle Antenna Search algorithm.

3. To introduce the type of beam to column connections, whether rigid or pinned, as a design variable.
4. To take into consideration the geometric constraints as part of the design optimization constraints.
5. To take into consideration the stability constraints as part of the design optimization constraints.

References

Aisc.org. (2017). American Institute of Steel Construction shapes database, 15th Edition. Retrieved: September 14, 2021, from: https://www.aisc.org/globalassets/aisc/manual/v15.0-shapes-database/aisc-shapes-database-v15.0.xlsx

Al-Mosawi, S. & Saka, M.P. (2000). Optimum shape design of cold-formed thin-walled steel sections. *Advances in Engineering Software*, vol. 31, no. 11, pp. 851-862.

American Institute of Steel Construction (AISC). (2016). *Load and Resistance Factor Design - Specification for Structural Steel Buildings (AISC 360-16).* AISC, Chicago, IL.

American Society of Civil Engineers (ASCE). (2016). *Minimum Design Loads and Associated Criteria for Buildings and other Structures (ASCE/SEI 7-16).* Reston, VA.

Aydogdu, I., Akin, A., & Saka, M.P. (2016). Design optimization of real-world steel space frames using artificial bee colony algorithm with Levy flight distribution. *Advances in Engineering Software*, vol. 92, pp. 1-14.

Camp, C.V., & Bichon, B.J. (2004). Design of space trusses using ant colony optimization. *Journal of Structural Engineering*, vol. 130, no. 5, pp. 741-751.

Camp, C.V., Bichon, B.J., & Stovall, S.P. (2005). Design of steel frames using ant colony optimization. *Journal of Structural Engineering*, vol. 131, no.3, pp. 369-379.

Carbas, S. (2016). Design optimization of steel frames using an enhanced firefly algorithm. *Engineering Optimization*, vol. 48, pp. 2007-2025.

Computers and Structures Incorporation (CSI). (2011). *Introductory Tutorial for SAP2000: Linear and Nonlinear Static and Dynamic Analysis and Design of Three-Dimensional Structures.* Version 15. California, USA.

Dogan, E., & Saka, M.P. (2012). Optimum design of unbraced steel frames to LRFD-AISC using particle swarm optimization. *Advances in Engineering Software*, vol. 46, pp. 27-34.

Hasancebi, O., Carbas, S., Dogan, E., Erdal, F., & Saka, M.P. (2010). Comparison of non-deterministic search techniques in the optimum design of real size steel frames. *Computers and Structures*, vol. 88, no.17-18, pp. 1033-1048.

Jiang, X., & Li, S. (2018). BAS: Beetle Antenna Search Algorithm for Optimization Problems. *International Journal of Robotics and Control*, vol. 1, no.1, pp. 1-5.

Latif, M.A., & Saka, M.P. (2019). Optimum design of tied-arch bridges under code requirements using enhanced artificial bee colony algorithm. *Advances in Engineering Software*, vol. 135.

Saka, M.P. (1991). Optimum design of steel frames with stability constraints. *Computers and Structures*, vol. 41, no. 6, pp. 1365-1377.

Saka, M.P. (2009). Optimum design of steel sway frames to BS 5950 using harmony search algorithm. *Journal of Constructional Steel Research*, vol. 65, no. 1, pp. 36-43.

Saka, M.P., & Geem, Z.W. (2013). Mathematical and metaheuristic applications in design optimization of steel frame structures: an extensive review. *Mathematical Problems in Engineering*, vol. 2013, pp. 1-33.

Saka, M.P., & Kameshki, E.S. (1998a). Optimum design of unbraced rigid frames. *Computers and Structures,* vol. 69, pp. 433-442.

Saka, M.P., & Kameshki, E.S. (1998b). Optimum design of nonlinear elastic framed domes. *Advances in Engineering Software,* vol. 20, no. 7-9, pp. 519-528.

Segui, W.T. (2007). *Steel Design*. Fourth Edition. Toronto, Canada: Thomson.

Thearchitectsdiary.com. (2020). Retrieved: September 11, 2022, from: https://thearchitectsdiary.com/the-10-best-benefits-of-steel-buildings-in-construction/

Williams, A. (2011). *Steel Structures Design: ASD/LRFD.* NJ: McGraw-Hill.

Yousif, S., & Saka, M.P. (2021). Enhanced beetle antenna search: a swarm intelligence algorithm. *Asian Journal of Civil Engineering*, vol. 22, pp. 1185-1219.

Appendix A

W-sections pool

Appendix A W-sections pool

This appendix presents the pool of W-sections and some of their section properties, that were used in the design optimization of steel space frames.

No.	Designation	Weight (kg/m)	Area (mm^2)	Depth (mm)
1	W150X18	18.0	2290	153
2	W100X19.3	19.3	2470	106
3	W200X19.3	19.3	2480	203
4	W310X21	21.0	2680	302
5	W250X22.3	22.3	2850	254
6	W200X22.5	22.5	2860	206
7	W310X23.8	23.8	3040	305
8	W130X23.8	23.8	3040	127
9	W150X24	24.0	3060	160
10	W250X25.3	25.3	3220	257
11	W200X26.6	26.6	3390	207
12	W130X28.1	28.1	3590	131
13	W310X28.3	28.3	3590	310
14	W250X28.4	28.4	3630	259
15	W150X29.8	29.8	3790	157
16	W200X31.3	31.3	3970	210
17	W310X32.7	32.7	4180	312
18	W250X32.7	32.7	4190	259
19	W360X32.9	32.9	4190	348
20	W200X35.9	35.9	4570	201
21	W150X37.1	37.1	4740	162
22	W250X38.5	38.5	4910	262
23	W310X38.7	38.7	4940	310
24	W410X38.8	38.8	4950	399
25	W360X39	39.0	4960	353
26	W200X41.7	41.7	5320	205
27	W360X44	44.0	5710	351
28	W310X44.5	44.5	5670	312
29	W250X44.8	44.8	5700	267
30	W410X46.1	46.1	5890	404
31	W250X49.1	49.1	6260	247

32	W360X51	51.0	6450	356
33	W460X52	52.0	6650	450
34	W310X52	52.0	6650	318
35	W200X52	52.0	6650	206
36	W410X53	53.0	6840	404
37	W360X57.8	57.8	7230	358
38	W250X58	58.0	7420	252
39	W200X59	59.0	7550	210
40	W310X60	60.0	7550	302
41	W460X60	60.0	7610	455
42	W410X60	60.0	7610	406
43	W360X64	64.0	8130	348
44	W530X66	66.0	8390	526
45	W310X67	67.0	8450	307
46	W410X67	67.0	8580	409
47	W250X67	67.0	8580	257
48	W460X68	68.0	8710	460
49	W200X71	71.0	9100	216
50	W360X72	72.0	9100	351
51	W250X73	73.0	9290	254
52	W310X74	74.0	9420	310
53	W530X74	74.0	9480	528
54	W460X74	74.0	9480	457
55	W410X75	75.0	9480	414
56	W360X79	79.0	10100	353
57	W310X79	79.0	10100	307
58	W250X80	80.0	10200	257
59	W610X82	82.0	10500	599
60	W530X82	82.0	10500	528
61	W460X82	82.0	10500	460
62	W530X85	85.0	10800	536
63	W410X85	85.0	10800	417
64	W310X86	86.0	11000	310
65	W200X86	86.0	11000	222
66	W460X89	89.0	11400	462
67	W250X89	89.0	11400	259
68	W360X91	91.0	11500	353

69	W610X92	92.0	11700	602
70	W530X92	92.0	11800	533
71	W460X97	97.0	12300	467
72	W410X100	100	12600	414
73	W200X100	100	12700	229
74	W250X101	101	12800	264
75	W530X101	101	12900	536
76	W360X101	101	12900	356
77	W610X101	101	13000	602
78	W460X106	106	13500	470
79	W310X107	107	13600	312
80	W530X109	109	13900	538
81	W360X110	110	14100	361
82	W460X113	113	14400	462
83	W610X113	113	14500	607
84	W410X114	114	14600	419
85	W250X115	115	14600	269
86	W310X117	117	15000	315
87	W360X122	122	15500	363
88	W530X123	123	15700	544
89	W690X125	125	15900	678
90	W610X125	125	15900	612
91	W460X128	128	16300	467
92	W310X129	129	16500	318
93	W250X131	131	16800	274
94	W410X132	132	16900	427
95	W760X134	134	17000	749
96	W530X138	138	17600	549
97	W690X140	140	17800	683
98	W610X140	140	17900	617
99	W310X143	143	18200	323
100	W460X144	144	18400	472
101	W760X147	147	18700	754
102	W250X149	149	18900	282
103	W410X149	149	19000	432
104	W530X150	150	19200	544
105	W690X152	152	19400	688

106	W610X153	153	19500	622
107	W610X155	155	19800	612
108	W460X158	158	20100	475
109	W310X158	158	20100	328
110	W760X161	161	20500	757
111	W360X162	162	20600	363
112	W530X165	165	21000	546
113	W250X167	167	21200	290
114	W690X170	170	21700	693
115	W760X173	173	22100	762
116	W610X174	174	22200	617
117	W840X176	176	22400	836
118	W460X177	177	22600	483
119	W310X179	179	22700	333
120	W360X179	179	22800	368
121	W530X182	182	23200	551
122	W760X185	185	23500	767
123	W690X192	192	24400	701
124	W840X193	193	24700	841
125	W460X193	193	24700	490
126	W610X195	195	24900	622
127	W760X196	196	25000	770
128	W530X196	196	25000	554
129	W360X196	196	25000	373
130	W920X201	201	25700	904
131	W310X202	202	25700	340
132	W840X210	210	26800	846
133	W460X213	213	27100	495
134	W360X216	216	27500	376
135	W610X217	217	27700	627
136	W690X217	217	27900	696
137	W530X219	219	27900	561
138	W760X220	220	28100	780
139	W1000X222	222	28300	970
140	W920X223	223	28600	912
141	W310X226	226	28800	348
142	W840X226	226	29000	851

143	W460X235	235	29900	500
144	W360X237	237	30100	381
145	W920X238	238	30300	914
146	W690X240	240	30700	701
147	W610X241	241	30800	635
148	W530X248	248	31500	572
149	W1000X249	249	31800	980
150	W840X251	251	31900	859
151	W920X253	253	32300	919
152	W310X253	253	32300	356
153	W760X257	257	32800	772
154	W460X260	260	33200	508
155	W610X262	262	33400	640
156	W360X262	262	33400	386
157	W690X265	265	33900	706
158	W920X271	271	34600	922
159	W1000X272	272	34400	991
160	W530X272	272	34600	577
161	W310X283	283	36100	366
162	W760X284	284	36200	780
163	W610X285	285	36500	648
164	W460X286	286	36300	518
165	W360X287	287	36600	394
166	W920X289	289	36800	927
167	W690X289	289	36800	714
168	W1000X296	296	37900	983
169	W840X299	299	38100	856
170	W530X300	300	38300	584
171	W610X307	307	39200	653
172	W920X313	313	39900	932
173	W310X313	313	39900	373
174	W360X314	314	40000	399
175	W1000X314	314	40100	1000
176	W760X314	314	40200	785
177	W460X315	315	40200	526
178	W1000X321	321	41000	991
179	W690X323	323	41200	721

180	W840X329	329	42100	861
181	W530X332	332	42900	594
182	W610X341	341	43400	660
183	W310X342	342	43700	384
184	W1100X343	343	43700	1090
185	W920X344	344	44000	927
186	W920X345	345	43900	943
187	W360X347	347	44200	406
188	W460X349	349	44300	536
189	W1000X350	350	44600	1010
190	W760X350	350	44700	795
191	W690X350	350	44800	729
192	W840X359	359	45900	869
193	W920X368	368	46800	932
194	W530X369	369	47600	602
195	W1000X371	371	47400	1000
196	W610X372	372	47400	668
197	W310X375	375	47800	391
198	W920X381	381	48600	951
199	W360X382	382	48800	417
200	W460X384	384	49000	546
201	W690X384	384	49100	737
202	W760X389	389	49700	803
203	W1100X390	390	49800	1100
204	W920X390	390	49800	937
205	W840X392	392	49900	876
206	W1000X393	393	49900	1020
207	W530X409	409	52800	612
208	W1000X412	412	52600	1010
209	W610X415	415	52800	678
210	W310X415	415	52800	404
211	W1000X415	415	53100	1020
212	W690X419	419	53600	744
213	W920X420	420	53500	942
214	W460X421	421	53700	556
215	W360X421	421	53700	424
216	W1100X433	433	55100	1110

217	W840X433	433	55200	884
218	W760X434	434	55500	813
219	W1000X438	438	55600	1030
220	W1000X443	443	56300	1010
221	W920X449	449	57400	947
222	W310X454	454	57700	414
223	W610X455	455	57900	688
224	W690X457	457	58200	752
225	W360X463	463	59000	434
226	W460X464	464	59100	566
227	W840X473	473	60500	894
228	W1000X483	483	61500	1020
229	W760X484	484	61900	823
230	W1000X486	486	61900	1040
231	W920X491	491	62500	958
232	W1000X494	494	63000	1040
233	W610X498	498	63400	699
234	W1100X499	499	63500	1120
235	W310X500	500	63800	427
236	W690X500	500	64000	762
237	W360X509	509	65200	445
238	W840X527	527	67100	904
239	W760X531	531	67700	833
240	W920X537	537	68400	965
241	W1000X539	539	68400	1030
242	W690X548	548	70300	772
243	W610X551	551	70300	711
244	W360X551	551	70300	455
245	W1000X554	554	71000	1030
246	W840X576	576	73500	914
247	W760X582	582	74200	843
248	W1000X584	584	74800	1060
249	W920X588	588	74800	975
250	W1000X591	591	75500	1040
251	W360X592	592	75500	465
252	W360X634	634	80600	475
253	W1000X642	642	81900	1050

254	W920X656	656	83900	988
255	W360X677	677	86500	483
256	W920X725	725	92300	998
257	W360X744	744	94800	498
258	W1000X748	748	95500	1070
259	W920X787	787	101000	1010
260	W690X802	802	103000	826
261	W360X818	818	105000	513
262	W1000X883	883	112000	1090
263	W360X900	900	115000	531
264	W920X970	970	124000	1040
265	W1000X976	975	125000	1110
266	W360X990	990	126000	549
267	W920X1077	1080	137000	1060
268	W360X1086	1090	139000	569
269	W920X1194	1190	152000	1080
270	W360X1202	1200	154000	579
271	W920X1269	1270	162000	1090
272	W360X1299	1300	166000	599
273	W920X1377	1380	175000	1090

Appendix B

OAPI Applications

Appendix B OAPI Applications

In this appendix, some of the applications of OAPI used in the design optimization of steel space frames are presented.

B.1. Open the Existing Model

```
AttachToInstance = false();
SpecifyPath = false();

ProgramPath = 'C:\Program Files\Computers and
Structures\SAP2000 23\SAP2000.exe';

APIDLLPath = 'C:\Program Files\Computers and
Structures\SAP2000 23\SAP2000v1.dll';

ModelDirectory = 'C:\Users\Admin\Desktop\CENG 599\8-
SAP 2000 - OAPI\Ahmed model - 105 member';

if ~exist(ModelDirectory, 'dir')
    mkdir(ModelDirectory);
end

ModelName = 'Tower5.sdb';

ModelPath = strcat(ModelDirectory, filesep,
ModelName);

%% create API helper object

a = NET.addAssembly(APIDLLPath);
helper = SAP2000v1.Helper;
helper =NET.explicitCast(helper,'SAP2000v1.cHelper');

if AttachToInstance
    SapObject =
helper.GetObject('CSI.SAP2000.API.SapObject');
    SapObject =
NET.explicitCast(SapObject,'SAP2000v1.cOAPI');
else
    if SpecifyPath
        %% create an instance of the SapObject from
the specified path
        SapObject = helper.CreateObject(ProgramPath);

    else
```

```
%% create an instance of the SapObject from the
latest installed SAP2000

        SapObject =
helper.CreateObjectProgID('CSI.SAP2000.API.SapObject'
);
    end
    SapObject =
NET.explicitCast(SapObject,'SAP2000v1.cOAPI');

%% start Sap2000 application

SapObject.ApplicationStart;

end
helper = 0;

%% create SapModel object

SapModel =
NET.explicitCast(SapObject.SapModel,'SAP2000v1.cSapMo
del');

%% initialize model

ret = SapModel.InitializeNewModel;
File
=NET.explicitCast(SapModel.File,'SAP2000v1.cFile');
ret = File.OpenFile(ModelPath);
```

B.2. Set Units, Save the Model and Run Analysis

```
%% set units to N_mm_C
ret=SapModel.SetPresentUnits(SAP2000v1.eUnits.N_mm_C)
;

%% save model
ret = File.Save(ModelPath);

 %% run model (this will create the analysis model)
Analyze =
NET.explicitCast(SapModel.Analyze,'SAP2000v1.cAnalyze
');

ret = Analyze.RunAnalysis();
```

B.3. Get Displacement Analysis Results

```
AnalysisResults =
NET.explicitCast(SapModel.Results,'SAP2000v1.cAnalysi
sResults');

AnalysisResultsSetup =
NET.explicitCast(AnalysisResults.Setup,'SAP2000v1.cAn
alysisResultsSetup');
    ret =
AnalysisResultsSetup.DeselectAllCasesAndCombosForOutp
ut;
  ret =
AnalysisResultsSetup.SetComboSelectedForOutput('SLS-
2');

    Disp_SLS1=zeros(54,6);   %54 is number of joints

for i = 1:54    % 54 is number of joints

    NumberResults = 0;
    Obj = NET.createArray('System.String',2);
    Elm = NET.createArray('System.String',2);
    Loadcase = NET.createArray('System.String',2);
    StepType = NET.createArray('System.String',2);
    StepNum = NET.createArray('System.Double',2);
    U1 = NET.createArray('System.Double',2);
    U2 = NET.createArray('System.Double',2);
    U3 = NET.createArray('System.Double',2);
    R1 = NET.createArray('System.Double',2);
    R2 = NET.createArray('System.Double',2);
    R3 = NET.createArray('System.Double',2);
       [ret, NumberResults, Obj, Elm, Loadcase,
StepType, StepNum, U1, U2, U3, R1, R2, R3] =
AnalysisResults.JointDispl(int2str(i),
SAP2000v1.eItemTypeElm.ObjectElm, NumberResults, Obj,
Elm, Loadcase, StepType, StepNum, U1, U2, U3, R1, R2,
R3);

Disp_SLS1(i,1)=double(U1);
Disp_SLS1(i,2)=double(U2);
Disp_SLS1(i,3)=double(U3);
Disp_SLS1(i,4)=double(R1);
Disp_SLS1(i,5)=double(R2);
Disp_SLS1(i,6)=double(R3);
end
```

B.4. Unlock the Model and Set Section

```
%% Unlocking the model
  ret = SapModel.SetModelIsLocked(false);

%% Set section to group 1
 FrameObj =
NET.explicitCast(SapModel.FrameObj,'SAP2000v1.cFrameO
bj');
 ret=FrameObj.SetSection('GROUP1', newG1section,
SAP2000v1.eItemType.Group);

 %% Set section to group 2
 FrameObj =
NET.explicitCast(SapModel.FrameObj,'SAP2000v1.cFrameO
bj');
 ret=FrameObj.SetSection('GROUP2', newG2section,
SAP2000v1.eItemType.Group);

  %% Set section to group 3
 FrameObj =
NET.explicitCast(SapModel.FrameObj,'SAP2000v1.cFrameO
bj');
 ret=FrameObj.SetSection('GROUP3', newG3section,
SAP2000v1.eItemType.Group);

  %% Set section to group 4
 FrameObj =
NET.explicitCast(SapModel.FrameObj,'SAP2000v1.cFrameO
bj');
 ret=FrameObj.SetSection('GROUP4', newG4section,
SAP2000v1.eItemType.Group);

  %% Set section to group 5
 FrameObj =
NET.explicitCast(SapModel.FrameObj,'SAP2000v1.cFrameO
bj');
 ret=FrameObj.SetSection('GROUP5', newG5section,
SAP2000v1.eItemType.Group);

  %% Set section to group 6
 FrameObj =
NET.explicitCast(SapModel.FrameObj,'SAP2000v1.cFrameO
bj');
 ret=FrameObj.SetSection('GROUP6', newG6section,
SAP2000v1.eItemType.Group);
```

```
   %% Set section to group 7
 FrameObj =
NET.explicitCast(SapModel.FrameObj,'SAP2000v1.cFrameO
bj');
 ret=FrameObj.SetSection('GROUP7', newG7section,
SAP2000v1.eItemType.Group);

   %% Set section to group 8
 FrameObj =
NET.explicitCast(SapModel.FrameObj,'SAP2000v1.cFrameO
bj');
 ret=FrameObj.SetSection('GROUP8', newG8section,
SAP2000v1.eItemType.Group);

    %% Set section to group 9
 FrameObj =
NET.explicitCast(SapModel.FrameObj,'SAP2000v1.cFrameO
bj');
 ret=FrameObj.SetSection('GROUP9', newG9section,
SAP2000v1.eItemType.Group);

    %% Set section to group 10
 FrameObj =
NET.explicitCast(SapModel.FrameObj,'SAP2000v1.cFrameO
bj');
 ret=FrameObj.SetSection('GROUP10', newG10section,
SAP2000v1.eItemType.Group);
```

B.5. Save the Model and Close the SAP 2000

```
%% save model
 ret = File.Save(ModelPath);

%% close Sap2000
ret = SapObject.ApplicationExit(false());
File = 0;
PropMaterial = 0;
PropFrame = 0;
FrameObj = 0;
PointObj = 0;
View = 0;
LoadPatterns = 0;
Analyze = 0;
AnalysisResults = 0;
AnalysisResultsSetup = 0;
SapModel = 0;
SapObject = 0;
```

الملخص

في هذه الأطروحة، يتم تنفيذ التصميم الأمثل للإطارات الفولاذية ثلاثية الأبعاد ذات الوصلات الصلبة بحسب كود المعهد الأمريكي للإنشاءات الفولاذية (LRFD-AISC). الخوارزمية التي تم تطويرها تجمع بين طريقة المعايير الأمثل (إحدى الطرق الحتمية) وخوارزمية بحث قرون استشعار الخنفساء (إحدى خوارزميات الأدلة العليا). يتم استيفاء شروط التصميم بواسطة الخوارزمية، بالإضافة لتقليل الوزن الإجمالي للإطار الفولاذي. يتم تحديد الأبعاد المقطعية عن طريق توصيف العضو الفولاذي (W) من القائمة الأمريكية المتاحة. يتم نمذجة الإطارات الفولاذية باستخدام برنامج SAP2000. تم ترميز معادلات أحكام التصميم بحسب الكود الأمريكي في برنامج MATLAB. كما تم ترميز معادلات الخوارزمية في البرنامج ذاته. تم الربط بين برنامجي SAP2000 وMATLAB باستخدام واجهة برمجة التطبيقات المفتوحة (OAPI). تم عرض نتائج التصميم الأمثل من خلال تطبيق الخوارزمية المطورة على إطار فولاذي مكون من طابق واحد، بالإضافة لإطار فولاذي آخر مكون من خمسة طوابق (تم اختياره من ورقة بحث سابقة). في هذه الأطروحة، تم تقليل عدد التكرارات (التحليل الهيكلي) بشكل كبير مقارنة بالطرق الأخرى التي تعتمد على خوارزمية الأدلة العليا فقط. بمعنى آخر، فقد أظهرت الخوارزمية المطورة أنها فعالة للحصول على التصميم الأمثل السريع للإطارات الفولاذية ثلاثية الأبعاد ذات الوصلات الصلبة.

جامعة البحرين

كلية الهندسة



قسم الهندسة المدنية

التصميم الأمثل السريع للإطارات الفولاذية ثلاثية الأبعاد ذات الوصلات الصلبة بحسب كود المعهد الأمريكي للإنشاءات الفولاذية

أطروحة مقدمة كجزء من متطلبات الحصول على درجة
الماجستير في الهندسة المدنية

إعداد

أحمد تحسين أحمد شناعه

الرقم الجامعي: ٢٠١٠٥٤٩٠

إشراف

د. سميح إرهان
أستاذ مساعد
جامعة البحرين

د. منى عبد اللطيف
أستاذ مساعد
جامعة البحرين

مملكة البحرين
فبراير، ٢٠٢٣م