UNIVERSITY OF GAZİANTEP
GRADUATE SCHOOL OF
NATURAL & APPLIED SCIENCES

GENETIC ALGORITHM OPTIMIZATION OF SPACE FRAMES

M. Sc. THESIS
IN
CIVIL ENGINEERING

BY
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JANUARY 2013

Genetic Algorithm Optimization of Space Frames

M. Sc. Thesis
In
Civil Engineering
University of Gaziantep

Supervisor
Asst. Prof. Dr. Nildem Tayşi

By
Ghedan Hussein
January 2013

T.C.
UNIVERSITY OF GAZİANTEP
GRADUATE SCHOOL OF
NATURAL & APPLIED SCIENCES
CIVIL ENGINEERING DEPARTMENT

Name of the thesis: Genetic Algorithm Optimization of Space Frame

Name of the student: Ghedan Hussein
Exam date: 28.January.2013

Approval of the Graduate School of Natural and Applied Sciences

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This is to certify that we have read this thesis and that in our opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

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Ghedan HUSSEIN

ABSTRACT

GENETIC ALGORITHM OPTIMIZATION OF SPACE FRAME

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M. Sc. in Civil Engineering
Supervisor: Asst. Prof. Dr. Nildem TAYŞI
January 2013, 74 pages

Structural design of space frames requires appropriate form for a structure so that it can carry the imposed loads safely and economically. Traditional approaches towards the task of finding such forms for structures have been by the use of experimental models or by intuition and experience. The main objective of this thesis is to develop and use reliable, creative and efficient computational tools for the linearly elastic analysis and optimum design of space frame structures under static loads.

The use of SAP2000 can assist greatly in achieving a safe design. However, commercially available programs are not designed as optimization tools. In this study for optimization of multistory structures, home written MATLAB code interface program is designed to connect SAP2000 which is known as a commercial nonlinear finite element program and genetic algorithm optimization program.

The design algorithm obtains minimum weight frames by selecting suitable sections from specified group list, with consideration actual design constraints like, strength, lateral displacement, inter story drift according to Load and Resistance Factor Design (LRFD). The improved method is tested on different three dimensional multi story moment resisting frames. It is concluded that this method can be used as a useful tool in engineering design and optimization.

Keywords: Optimum design, Genetic algorithm, Steel structure, SAP2000, OAPI.

ÖZET

UZAY ÇERÇEVELERİN
GENETİK ALGORİTMA İLE OPTİMİZASYONU

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Ocak 2013, 74 sayfa

Uzay çerçevelerin tasarımı, uygulanan yükü güvenli ve ekonomik bir şekilde taşıyabilmesi için, uygun bir yapı formu gerektirir. Bu formlar bugüne kadar genellikle geleneksel yöntemler olan deneysel modellerin veya deneyimlerin sonucunda bulunmaktadır. Bu tezin ana amacı, statik yükler altındaki uzay çerçeve yapıların analizi ve optimum tasarımı için güvenilir, yaratıcı ve etkili bir bilgisayar programı geliştirip kullanmaktır.

SAP2000 programı güvenli binalar tasarlamak için büyük oranda yardımcı olmaktadır. Fakat bununla birlikte mevcut ticari programlar optimizasyon araçları olarak tasarlanmamışlardır. Bu çalışmada çok katlı yapıların optimizasyonu için, ticari lineer olmayan sonlu elemanlar programı olarak bilinen SAP2000 programı ile genetik algoritma optimizasyon programını, MATLAB kodlarını kullanarak birleştiren arayüz programlı geliştirilmiştir.

Tasarım algoritması, minimum ağırlıklı çerçeveleri, uygun kesit listelerinden seçerek, yük ve mukavemet faktörü ilkesi (LRFD)'ne göre, gerilme ve deplasman gibi gerçek tasarım kısıtlayıcılarını kullanarak elde eder. Geliştirilen yöntem farklı üç boyutlu, çok katlı çerçeve yapılar üzerinde test edilmiştir ve bu yöntemin mühendislik tasarımı ve optimizasyonu için kullanışlı bir yöntem olduğu sonucuna varılmıştır.

Anahtar kelimeler: Optimum tasarım, Genetik algoritma, Çelik yapılar, SAP2000, OAPI.

ACKNOWLEDGMENT

At the outset, I would like to profoundly thank Asst. Prof. Dr. Nildem TAYŞI for her guidance, encouragement and good advice. This thesis is much work better thanks for her supervision. It's worthy to mention that I've learned many valuable things since I became her student. She generously spent very much of her precious time instructing me how to write a paper, how to search for literature and how to collect data.

I'm also so grateful to Prof. Dr. Mustafa Özakça and Asst. Prof Dr. Ahmet Erkliğ for serving on the committee and for spending times read this thesis and provide useful suggestion to enrich the thesis.

I would like to thank my parents, my sisters and brothers for their helping, without their support, this research project would not have been possible.

During the period of two years, many friends are helpful to color my academic life. I would like to take a moment to deeply thank all of them.

Finally, without the support of my wife Galawezh and my children, Yara and Anya, I would never have succeeded. Although they often had to endure my absence they seldom complained. I hope they will enjoy the effort being completed.
Thanks forever.

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LIST OF SYMBOLS/ABBREVIATIONS

2D	Two Dimensional
3D	Three Dimensional
FE	Finite Element
GAs	Genetic Algorithms
AISC	American Steel Construction Manual Constitute
ASD	Allowable Stress Design
LRFD	Load and Resistance Factor Design
OAPI	Open Application Programming Interface
CS	Cuckoo Search
DOF	Degrees Of Freedom
LP	Linear Programming
NLP	Nonlinear Programming
SE	Strain Energy
n_{pop}	Population size
MDB	Data base file
XLS	Microsoft Excel
TXT	Text file

Scalar

A	area
D	global displacement

Ag	cross-sectional area of a member
d_{all}	allowable displacement
d_1-d_2	Physical deformation
d_{max}	greatest permissible displacement
J	polar moment of inertia
E	Young’s modulus
$F(\mathbf{x})$	objective function to be minimized
X	design variable
G	modulus of rigidity
Fy	Yield stress of steel.
F_a	Axial compressive stress
F_b	Compressive bending stress that would be permitted if bending moment alone existed
F_{EX}	Euler stress divided by a factor of safety
f_a	Computed axial stress
f_b	Computed compressive bending stress at the point under consideration
Cm	Coefficient whose value is taken as 0.85 for compression members in frames subject to sidesway
f_{bt}	computed bending tensile stress
f_{at}	computed axial tensile stress
F_{bt}	allowable bending stress
F_t	governing allowable tensile stress

$g_j(\mathbf{x})$	inequality constraint function
$h_k(\boldsymbol{x})$	equality constraint function
$\hat{f}(\mathbf{s})$	fitness function
x_i	*ith* design variable
s_k^l	lower bounds on a typical design variable
s_k^u	upper bounds on a typical design variable
k	quantity of design variables
p_i	probability of selected *ith* string
P_u	required axial strength (tension or compression)
P_n	nominal axial strength (tension or compression)
M_{ux}	required flexural strength about the major axis
M_{uy}	required flexural strength about the minor axis
M_{nx}	nominal flexural strength about the major axis
M_{ny}	nominal flexural strength about the minor axis
I	moment of inertia
K	effective length factor
ℓ	Length
γ	shear strain
u	axial deformation in the x direction
v	deflection in the y direction
u_ℓ ,, w_ℓ	displacement components in ℓ , y and n-directions
θζ	rotation in the x-y plane
u,w	global displacement parameters

m_e	element mass matrix for the frame element
f_{sxn}	concentrated forces
m_{s1}	concentrated moments
f_e	total nodal force vector
D_{3i-2}	rotational deformation

Vector

$\mathbf{d_e}$	displacement vector based on local coordinate
$\mathbf{D_e}$	displacement vector based on global coordinate
$\boldsymbol{M_e}$	total global moment vector
$\boldsymbol{F_e}$	total global force vector

Matrix

$\mathbf{T}$	Transformation matrix
$\boldsymbol{K_e}$	global stiffness matrix

Greek Symbols:

Ø	angle between local and global axes
ϕ_t	resistance factor for tension
ϕ_c	resistance factor for compression
θ_{z2}	rotations of beam element in two nodes
α	angle between local and global axes, Newmark's Method modification factor
η_i	index number according to fabrication code
κ	shear modification factor
ρ	mass density

CHAPTER 1

INTRODUCTION

1.1 General

Structural design has always been a very interesting and creative segment in a large variety of engineering projects. Structures should be designed such that they can resist applied forces (stress constraints) and do not exceed certain deformations (displacement constraints) [1].

The development and validation of methods for obtaining optimal steel frame designs has merited significant attention for several decades. The objective of steel frame optimization is the minimization of the cost of frame design, subject typically to strength and serviceability constraints. The wide-flange shapes provided in the AISC steel construction manual constitute the variable space in steel optimizations. As steel shapes do not exist on a continuous scale of cross-sectional area, moment of inertia, or any other section parameter, frame optimization problems are typically conducted on discrete spaces, rendering deterministic gradient-based methods impractical. Also, as structural system response is the result of complicated interaction between various members, steel frame optimizations are also highly nonlinear. Despite these inherent difficulties, the development of innovative stochastic algorithms and increase in computing capability has enabled optimal designs for large, discrete structural optimization problems with various constraints to be obtained within reasonable computational expense.

The structural steel design is a process based on many contributing aspects such as; past successful and unsuccessful experience of construction, laboratory tests and search results. Structures can be used safely and efficiently but at the same time must be economically built and maintained therefore the design process must satisfy two conflicting aims safety and economy. Achieving this compromise is not easy, consequently codes of practice have evolved to assist and guide the designer. Different international codes, such as, American and British codes treat differently. This may be due to the methods of design are still at an elementary stage of development. So the problem of design is much less specific than analysis.

In this thesis more robust, improved and faster algorithm is presented for analysis and optimization of different space moment steel structures of practical interest. Wind, gravity loads, and stress constraints with total interstory drifts according to construction code requirements are considered. Other practicable structure constraints are considered by including grouped members; a unique AISC W-section is assigned to the entire member of groups. Also, variety of W-section or size can be limited, for reducing the variety of sections and number of parameters.

Genetic Algorithm (GA), interfaced with a SAP2000 commercial package program, is utilized to produce the optimal solutions. SAP2000 structure analysis program is a well-known integrated Finite Element (FE) structural analysis tool which already used for modelling and designing structures according to different design codes. The Open Application Programming Interface (OAPI) in SAP2000 is a free service in some versions of it, to export and import data files from and to SAP2000. In SAP2000 after input file being opened, SAP2000 will analyse, save result and design all members. From one of output files, any required data’s, like element stress and joint displacements can be found to check strength and serviceability constraints [2].

1.2 Thesis Objectives

The main objective of the thesis is to develop and demonstrate the use of a reliable, creative, efficient and competitive computational tool for the optimum design of space buildings under static loads.

Different kinds of meta-heuristic algorithms have been recently utilized to overcome the complex nature of optimum design of structures. In this thesis, an integrated optimization procedure of real size structures is simply performed interfacing SAP2000 and MATLAB® software's in the form of single computing. The meta-heuristic algorithm chosen here is GA.

The optimization refers to weight optimization according to different design codes. The requirement is that the algorithm only proposes frames that consists of elements taken from an available profiles list, and that it satisfies the relevant constraints given in different design codes, like American Institute of Steel Construction (AISC-LRFD) and (AISC-ASD) specifications.

Different space moment frames under strength constraints of the AISC-ASD and AISC-LRFD specifications, geometric limitations and displacement constraints are optimized. New integrated optimization procedure proposed involving the SAP2000 by its OAPI functions is validated on several inclusive steel structures and the results show that using the parallel computing besides the reliable SAP2000 analyser efficiently optimizes typical structural systems for practical purposes.

1.3 Layout of the Thesis

In present work main attention is focused on optimization of steel moment frame under wind and gravity loads. Stress constraints with total story drifts according to American construction code requirements are considered. For optimization the application of SAP2000 in the form of interfacing program is employed. The main goal of the study is to reduce the total weight in such way that they can resist applied forces and do not exceed certain deformations.

The organization of the study and the layout of the thesis is now pronounced:

Chapter 2 is the literature review in application of GA and SAP2000 in structural optimization field.

Chapter 3 illustrates the fundamental formulation for 2D and 3D analysis methods. The primary assumption as well as matrix analysis is also presented.

Chapter 4 deals with various aspects of the optimization process including the definition and selection of the design variables (cross sections) and the application of SAP2000 in structural optimization to help achieving safe design.

Chapter 5 this chapter deals with numerical applications of optimization examples of space moment frames.

Chapter 6, finally in this chapter some brief conclusions are presented together with some suggestions for future works.

CHAPTER 2

LITERATURE SURVEY

2.1 Introduction

Many mathematical programming methods have been developed during the last three decades [3, 4]. Most design applications in civil engineering involve selecting values for a set of design variables. Mathematically these design variables are the cross sectional area of structure members and they are discrete for most design problems. However, generally mathematical optimization applications are suitable for continuous design variables. In structural optimization with discrete design variables, searching for optimal solution becomes a difficult task. These methods include complete enumeration techniques. Different optimization methods and their development progresses as discussed in Chapter 4 were used by researchers to illustrate the performance of space moment frame structures under service loads. These techniques are also discussed in the different design codes to restrict the optimization techniques.

There are fascinating algorithms, the name came from the way in which they loosely mimic the process of evolution of organisms, where a problem solution stands in for organism's genetic string. Features include a survival of the fittest mechanism in which potential solutions in a population are pitted against each other, as well as recombination of solutions in mating process and random variations. The incredible part is that this heuristic can "evolve" better and better solutions without any deep understanding of the problem itself.

In this study GA technique for optimization design of frames using discrete structural elements are presented. The optimum design of space frame structures and the application of GA technique have found great attention in literature. In this section the literature is observed and classified with related studies work with GAs or interfacing with traditional method.

2.2 Structural Optimization Using Genetic Algorithms

GA, a member of evolutionary algorithm, is a population-based global search technique based on the Darwinian Theory [5]. The name came from the way in which they loosely mimic the process of evolution of organisms, where a problem solution stands in for organism's genetic string. Professor John Holland at the University of Michigan examined GA as search procedures based on natural selection and survival of the fittest [6].

GA is technique which finds the optimal solution through repetitive analysis. Therefore it has a lot of computational complexities and lacks the work applicability [7].

The recent decade scientific publications about application of GA grew at approximately 40 % and peaked in year 1995 [8]. David E. Goldberg (one of Professor John Holland's student) [9, 10] seems to be the first one used GA in structural optimization. In 1986, he and a graduate student of his used the GA technique to minimize the weight of a ten bar aluminium truss. This structure is commonly used as a benchmark problem in structural optimization [9].

Hajela in 1989 [11] seems to be the first one used GA to obtain minimum weight of two grillage beam under elastic design criteria; the structure was subjected to a uniformly distributed load. After that, Hajela (1990) [12] and Hajela and Lin [13] published several papers on the use of GAs in structural optimization. Also he presented an optimized 10-bar truss structure subjected to a sinusoidal load.

Genetic programming is an effective search technique based on natural selection. The basic idea is to combine good solutions to a certain problem over many generations to gradually improve the result. All solutions are initially created randomly, and they are individually represented by a binary string with some similarities to natural chromosomes, hence the name genetic programming [14].

As mentioned in different GA sources, GA operation requires very large computation time to converge. Therefore, it requires some modification in which a resizing algorithm module is embedded. Se Woon Choi et al [7] used modified GA by resizing algorithm model to reduce converge time. The author optimized 2D-3story moment space frame under member stress and total interstory drift constraints. He concluded that proposed GA has shown more rapid speed of convergence when compared to optimum design model using original GA before resizing the algorithm model.

2.3 Applications of Genetic Algorithm in Structural Optimization

In this section the literature is observed some of modified GAs in optimization of structural frames. The major application of GAs has been the automated design of steel frame structures and has followed several avenues. The first is topology and shape optimization, in which the applications have included elastic truss structures subjected to static loading [15]. The second and vast majority of this effort has been restricted to the optimized design of two dimensional (2D) moment frame [16].

Pezeshk et al [17] optimized two steel moment-frame structures using GAs. Optimizations were performed for linear, linear P-delta, and nonlinear P-delta structural analyse cases. The frames were optimized over a discrete variable domain of available wide-flange steel sections.

Hayalioglu [18] compared the relative efficiency of the LRFD and ASD codes in optimizing a trio of space frames using GAs. The frames were constrained by code specified design strength and maximum top-story drift requirements.

The discrete variables linked to the problem of optimal structural design were presented by Huang et al [19]. When selecting discontinues value for a variable, values must also be selected from a table with other variables linked to it. A major application area was such problems of the design steel structures using available sections. They made three approaches that link an optimization method of continuous variable with a simulated annealing, branch and bound method, and GA, are shown and applied in a computer program for their numerical evaluation. The design problems solution of three structural explained to study the performance of the suggested methods.

2.4 Optimization in the Form of Interfacing Program

The use of commercial nonlinear FE package which is specially designed for analysing of structure can assist vastly in achieving a safe design [2].

The main design effort involves sizing the individual beam, column, and bracing members after the topology and support conditions are established for a frame structure. Members are categorized into certain groups according to symmetry and fabrication conditions known as design variables. A new integrated optimization procedure proposed involving the SAP2000 by its OAPI functions is used. Using the parallel computing besides the reliable SAP2000 analyser efficiently optimizes typical structural systems for practical purposes.

Since the interfacing of commercial FE analysis programs with GA is possible, it will be a good chance for using commercial GA-FEM in the form of combination for research [20].

Mohammad Ghozi, et al [2, 20, 21] published three different papers on the using of GAs in structural optimization by interfacing an evolutionary algorithms (GA) and FE commercial structure analysis program.

In the first paper authors [2], optimized trusses, and compared the results with the other researches, with their method, optimization of 3 bench mark trusses has been solved by using the number of personal computer as a slave PC in the form of parallel and single computing to combine GA and SAP2000 in an open home written code

In the second paper authors [20] optimized 2D steel structure under dead, live and wind loads, by combining GA with SAP2000 under theories of strong column weak beam concept, concluded that strong column weak beam as constraint is useful and should be included in design of steel structure.

In the third paper authors [21] optimized 2D steel structures by combining GA and SAP2000 in an open visual basic code under concept of column failure mechanism. The constraints are consisting of fabrication conditions, imposed to group together the relative sizes of the member cross sections.

Kaveh et al [22] published an article about optimization of space frame using interfaced Cuckoo Search (CS) as a meta-heuristic algorithm with nonlinear finite element analysis program (SAP2000) in the form of parallel computing. The implemented method is tested on three different braced and un braced multi-storey space moment frames, concluded that parallel Computing technique is an effective time-saving procedure and the proposed method is effective in optimizing practical structures. Undeniably, SAP2000 program enables user to work with any other type of structures, international codes, load types and their combinations, as well as linear, nonlinear, static and dynamic analysis of structures.

CHAPTER 3

STATIC ANALYSIS OF STEEL SPACE FRAME

3.1 Introduction

Frame structures are commonly used in structural engineering applications in different forms as plane (2D) and space (3D) frames, which are made of steel, reinforced and pre-stressed concrete (RC) or timber [23]. Plane frames are composed of arbitrarily oriented beam elements joined together in a plane with distributed loading on elements and/or concentrated loads in the same plane.

Structural analysis, which is an integral part of any structural engineering project, is the process of predicting the performance of a given structure under a prescribed loading condition. The performance characteristics usually of interest in structural design are: (a) stresses or stress resultants (i.e., axial forces, shears, and bending moments); (b) deflections; and (c) support reactions. Thus, the analysis of a structure typically involves the determination of these quantities as caused by the given loads and/or other external effects (such as support displacements and temperature changes).

Space frame is the common to the most types of the framed structures which have members oriented in any direction in the 3D space Figure 3.1. The connections may be rigid or flexible between elements of the structure and external loads acted in any randomly directions and can apply to the joints and the members of space frames generally subject to bending moments, shears about both principle axes directions and axial force. The analysis of space frames is commonly based on the assumption that the cross-sections of all the members are symmetric about at least two mutually perpendicular axes, and are free to warp out of their planes under the action of torsion moments.

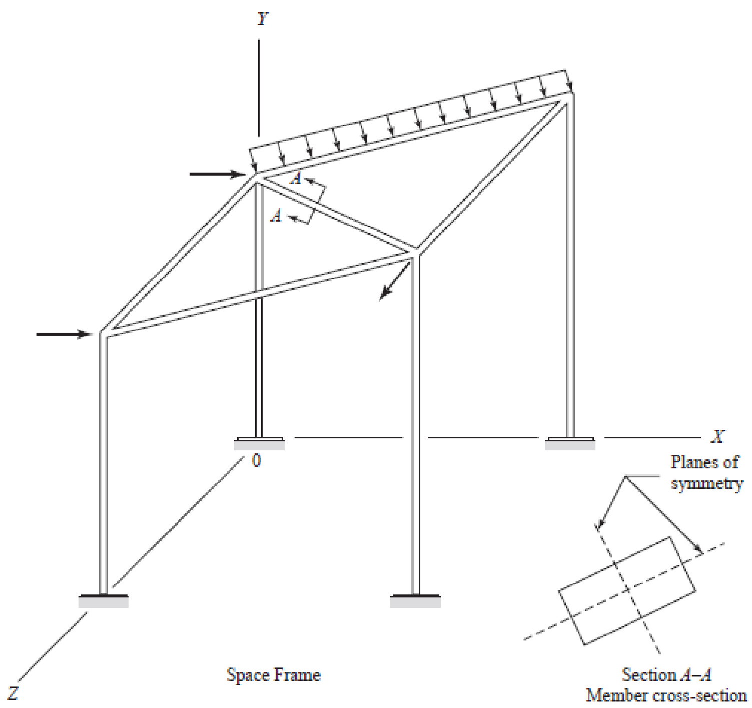


Figure 3.1 Space frame [24]

3.2 Analysis of Space Moment Frame

Before the computer technology is developed, they have been mostly used in practice by modelling a structural system and loading in different planes due to analysis simplicity. Today, since the capabilities and capacities of computer are at high level and still increasing, the need of simplification of structural system is not essential for calculation purposes and more realistic structural models are used by applying 3D beam elements from which 2D elements are obtained as special case [23].

Linear-elastic analysis of a rigidly-connected space frame system has been carried out by using the matrix displacement method. In rigid space frames each joint or node has 6 DOF and each member has 6 stress-resultants in the local coordinates. These are twisting moment or torque, two moments causing curvature in the (z-x) plane, two moments causing curvature in the (z–y) plane, and axial tension as shown in Figure 3.2 [18].

The number of degrees of freedom in the rigid frame element remains constant between both two global and local transformations axes. [25].

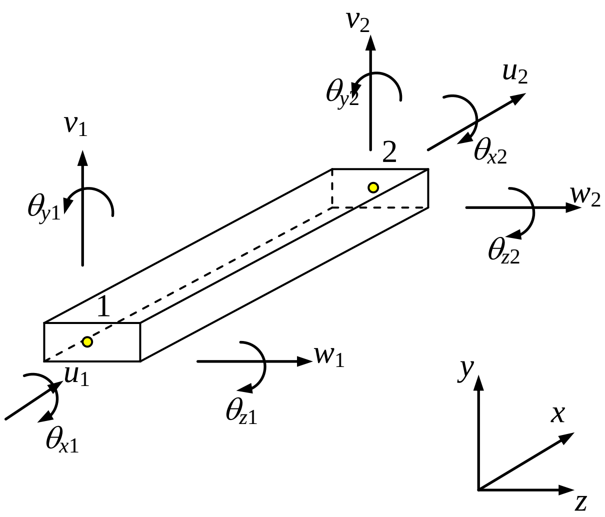


Figure 3.2 Frame element displaying local coordinate system

In general two beam theories are used in the framed structural analysis:

- Euler-Bernoulli beam theory.
- Timoshenko beam theory.

3.2.1 Euler-Bernoulli beam theory

Euler-Bernoulli theory is based on simplified linear theory of elasticity and used to calculate the load carrying and deflection characteristics of beam in general. The Euler Bernoulli theory, depends on assumptions that the planes normal to the midline remain plane and normal, it means that θ_1 and θ_2 equal μ_1 and μ_2 respectively [23] see Figure 3.3

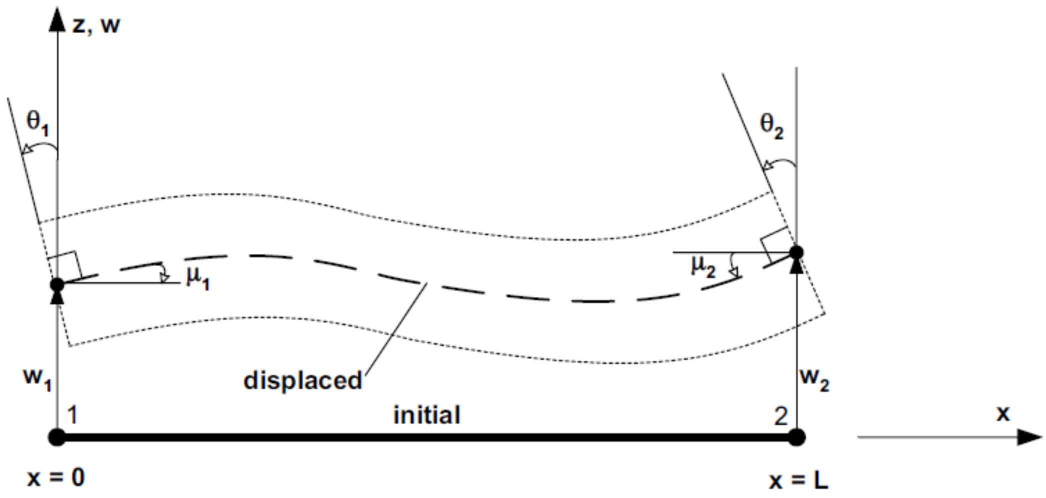


Figure 3.3 Euler-Bernoulli beam theory [26]

These assumptions are valid for long, slender and thin beams of isotropic materials with solid cross-sections. For short and thick beams the Euler-Bernoulli beam theory may be incorrect and misleading since the effect of the transverse shear deformation is not included in the transverse shear deformation at the first time.

As described by Timoshenko (1957) $\mu = dw/dx$ so the deflection equation for such a beam can be derived from the fact that $\theta = \mu$ and given as

$$\frac{d^2w}{dx^2} = -\frac{M}{EI} \tag{3.1}$$

By ignoring axial effects, the Strain Energy (SE) per unit length becomes

$$\frac{SE}{length} = \frac{1}{2}EI\left(\frac{d^2w}{dx^2}\right)^2 \tag{3.2}$$

3.2.2 Timoshenko beam theory

Timoshenko beam theory is applicable for both thick and thin beams and the Euler Bernoulli beam theory is obtained as a special case of the Timoshenko beam theory.

Timoshenko solved the neglected effect of the transverse shear deformation at the first time in the Euler-Bernoulli beam theory by introducing shear coefficient k as a correction factor to allow the non-uniform shear strain to be expressed as a constant, since according to Timoshenko beam theory's assumption the shear strain and stress are constant over the cross section The shear coefficient approximates the correct integrated value of SE due to shear ($1/2\tau\gamma$) as an assumed constant average or centreline value [26].This value depends on the shape of the cross section. The plane section still remains plane but rotates by an amount, θ, equal to the rotation of the neutral axis, μ, minus the shear strain γ Figure 3.4.

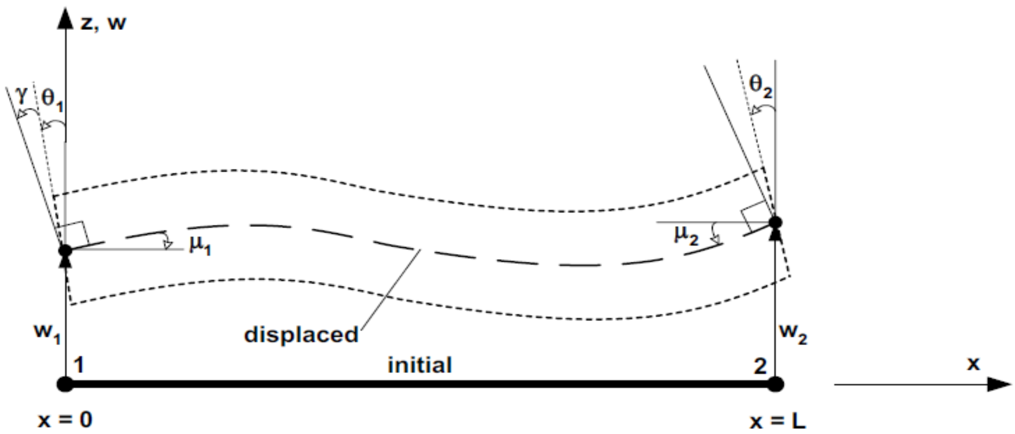


Figure 3.4 Timoshenko beam element [26]

$$\theta = \mu - \gamma \tag{3.3}$$

μ = dw/dx leads to

$$\frac{d\theta}{dx} = \frac{d^2w}{dx^2} - \frac{d\boldsymbol{\gamma}}{dx} \tag{3.4}$$

The SE per unit length is thus

$$\frac{SE}{length} = \frac{1}{2}EI\left(\frac{d^2\theta}{dx^2}\right)^2 + \frac{1}{2}kAG\gamma^2 \tag{3.5}$$

3.3 Finite Element Analysis of Frames

Finite Element (FE) Method is a numerical procedure for solving engineering problems; this method is widely method in almost all analysis disciplines. Its formulations are based on vibration principles.

The FE analysis consists of some principal steps like:

1. Discretizing the domain wherein each step involves subdivision of the structure into nodes and elements.

2. Making stiffness matrices for each element in the domain.

3. Arrangement the global stiffness matrix.

4. Applying boundary equation consists of specified, force, displacement, and support conditions.

5. Solving equation.

3.3.1 Planar frames

A plane frame is a 2D assemblage of straight members connected together by rigid and/or hinged connections, and subjected to loads and reactions that lie in the plane of the structure. Under the action of external loads, the members of a plane frame may be subjected to axial forces like the members of plane trusses, as well as bending moments and shears like the members of beams. Therefore, the stiffness relations for plane frame members can be conveniently obtained by combining the stiffness relations for plane truss and beam members [24].

3.3.1.1 FEM equations for planar frames in local coordinate system

In a 2D frame element, there are three DOF at each node in their local coordinate systems, which consist of deflection in y-direction *v*, axial deformation in the direction of x-axis, *u*, and the rotation in the x-y plane with respect to the z-axis, θζ. [27] as shown in Figure 3.5.

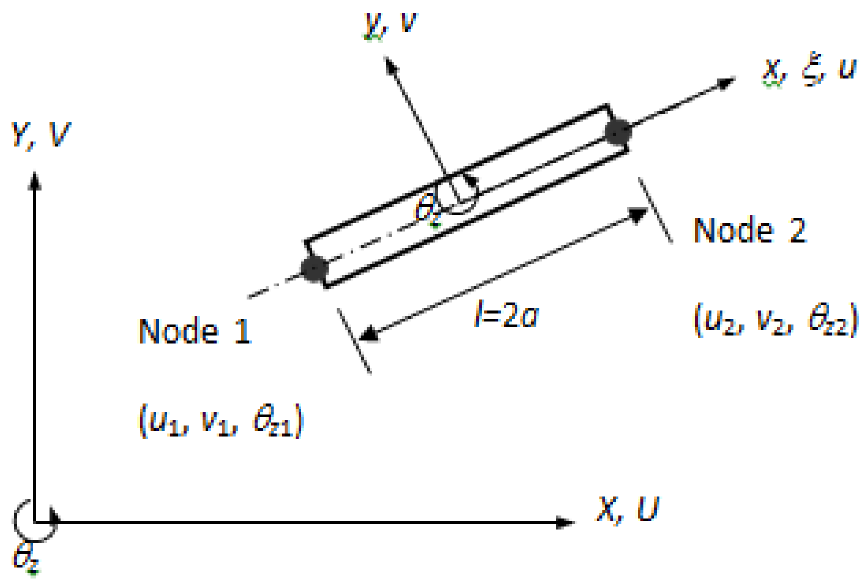


Figure 3.5 Planar frame element and the DOFs.

Combining DOF of truss which consist of axial deformation with DOF of beam element at each node (transverse deformation and rotation) will give the degree of freedom of a frame element.

$$\mathbf{d}_e = \begin{Bmatrix} u_1 \\ v_1 \\ \theta_{z1} \\ u_2 \\ v_2 \\ \theta_{z2} \end{Bmatrix} \tag{3.6}$$

Where u_1 and u_2 are truss axial deformation and v_1, v_2, θ_{z1} and θ_{z2} are transverse deformation and rotations of beam element in two nodes.

To obtain the stiffness matrix for truss elements is first extended to a 6 x 6 matrix depending on the number of DOF of the truss element in the element displacement matrix in Eq. (3.6):

$$\mathbf{k}_e truss = \begin{bmatrix} \frac{AE}{2a} & 0 & 0 & -\frac{AE}{2a} & 0 & 0 \\ & 0 & 0 & 0 & 0 & 0 \\ & & 0 & 0 & 0 & 0 \\ & & & \frac{AE}{2a} & 0 & 0 \\ & sy. & & & 0 & 0 \\ & & & & & 0 \end{bmatrix} \tag{3.7}$$

Also the stiffness matrix of beam element is extended to a 6 x 6 according to the order of the DOF of the beam element in Eq. (3.6)

$$\mathbf{k}_e beam = \begin{bmatrix} 0 & 0 & 0 & 0 & 0 & 0 \\ & \frac{3EI_z}{2a^3} & \frac{3EI_z}{2a^2} & 0 & -\frac{3EI_z}{2a^3} & \frac{3EI_z}{2a^2} \\ & & \frac{2EI_z}{a} & 0 & -\frac{3EI_z}{2a^2} & \frac{EI_z}{a} \\ & & & \frac{AE}{2a} & 0 & 0 \\ & sy. & & & \frac{3EI_z}{2a^3} & -\frac{3EI_z}{2a^2} \\ & & & & & \frac{2EI_z}{a} \end{bmatrix} \tag{3.8}$$

Combining equation (3.7) and (3.8) makes matrix for the frame element:

$$\mathbf{k}_e = \begin{bmatrix} \frac{AE}{2a} & 0 & 0 & -\frac{AE}{2a} & 0 & 0 \\ & \frac{3EI_z}{2a^3} & \frac{3EI_z}{2a^2} & 0 & -\frac{3EI_z}{2a^3} & \frac{3EI_z}{2a^2} \\ & & \frac{2EI_z}{a} & 0 & -\frac{3EI_z}{2a^2} & \frac{EI_z}{a} \\ & & & \frac{AE}{2a} & 0 & 0 \\ & sy. & & & \frac{3EI_z}{2a^3} & -\frac{3EI_z}{2a^2} \\ & & & & & \frac{2EI_z}{a} \end{bmatrix} \quad (3.9)$$

Same way as before to obtain element mass matrix of the frame element, by extending both truss and beam element mass matrixes ($\mathrm{m_e}$) into 6 x 6 matrices and added together.

$$\mathrm{m_e} = \rho A a / 105 \begin{bmatrix} 70 & 0 & 0 & 35 & 0 & 0 \\ & 78 & 22a & 0 & 27 & -13a \\ & & 8a^2 & 0 & 13a & -6a^2 \\ & & & 70 & 0 & 0 \\ & sy. & & & 78 & -22a \\ & & & & & 8a^2 \end{bmatrix} \quad (3.10)$$

The force vectors of both beam and truss elements are extended into 6 x 1 vectors and added together corresponding to their respective DOFs. In case when the element is subjected to external distributed loads *fx* and f_y along the x-axis; concentrated forces f_{sx1}, f_{sx2}, f_{sy1} and f_{sy2}; and concentrated moments m_{s1} and m_{s2}, respectively, at nodes 1 and 2, the total nodal force vector becomes.

$$f_e = \begin{Bmatrix} f_x a + f_{sx1} \\ f_y a + f_{sy1} \\ \dfrac{f_y a^2}{3} + m_{s1} \\ f_x a + f_{sx2} \\ f_y a + f_{sy2} \\ \dfrac{-f_y a^2}{3} + m_{s1} \end{Bmatrix} \tag{3.11}$$

3.3.1.2 Equations in global coordinate system

Formulated matrices in the previous section are for a particular frame element in a specific orientation. A full frame structure usually consists of numerous frame elements which are joined together in different orientations and these orientations are different in their local coordinate system. To collect the element matrices together, firstly all the matrices must be converted according to its global coordinate system.

Assume that local nodes 1 and 2 in Figure 3.6 correspond to the global nodes i and j, respectively. The displacement at a local node should have a rotational deformation with two translational components in the x and y directions (θ, u, v). Also two translational components in the X and Y directions and one rotational deformation for the displacement at a global node should have (D_{3i}, D_{3i-2} and D_{3i-1}) for the ith node.

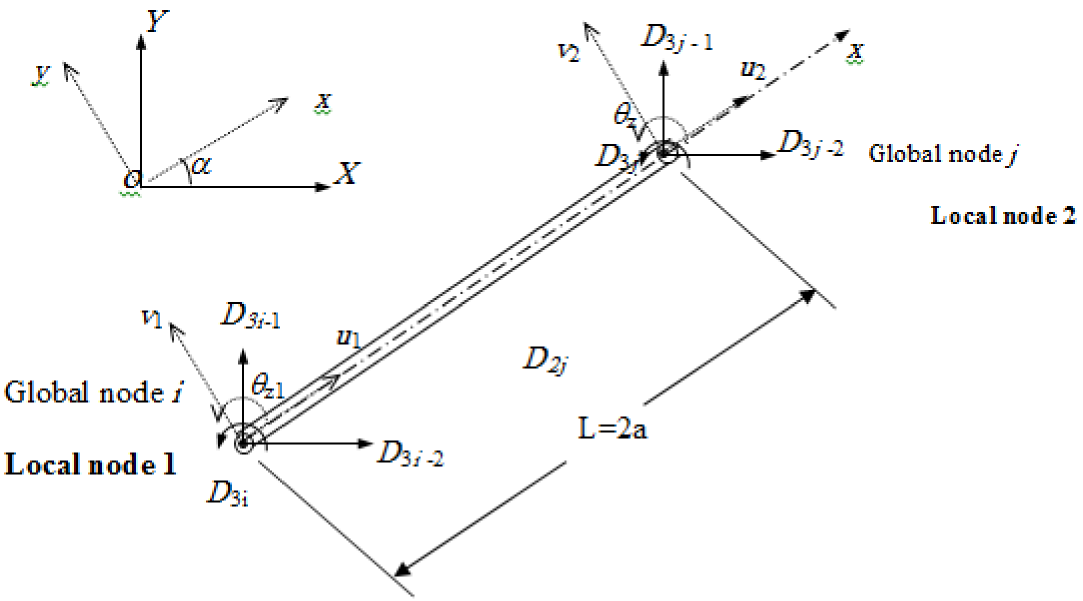


Figure 3.6 Coordinate transformation

The same sign convention also applies to node *j*. The coordinate transformation gives the relationship between the displacement vector $\mathbf{d_e}$ and the displacement vector $\mathbf{D_e}$ for the specified element, based on the global coordinate system.

$$\mathbf{d_e} = \mathbf{TD_e} \tag{3.12}$$

Where

$$\mathbf{D_e} = \begin{Bmatrix} D_{3i-2} \\ D_{3i-1} \\ D_{3i} \\ D_{3j-2} \\ D_{3j-1} \\ D_{3j} \end{Bmatrix} \tag{3.13}$$

and **T** is the transformation matrix for the frame element given

$$\mathbf{T} = \begin{bmatrix} l_x & m_x & 0 & 0 & 0 & 0 \\ l_y & m_y & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & l_x & m_x & 0 \\ 0 & 0. & 0 & l_y & m_y & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix} \tag{3.14}$$

Where

$$l_x = \cos(\mathrm{x}, \mathrm{X}) = \cos\alpha = \frac{X_{J-}X_i}{2a} \tag{3.15}$$

$$m_x = \cos(\mathrm{x}, \mathrm{Y}) = \cos\alpha = \frac{y_{J-}y_i}{2a} \tag{3.16}$$

$$l_y = \cos(\mathrm{y}, \mathrm{X}) = \cos(90 + \alpha) = -\sin\alpha = -\frac{y_{J-}y_i}{2a} \tag{3.17}$$

$$m_y = \cos(\mathrm{y}, \mathrm{Y}) = \cos\alpha = \frac{X_{J-}X_i}{2a} \tag{3.18}$$

$$2a = \frac{\sqrt{(X_{J-}X_i\ )^2 + (Y_{J-}Y_i\ )^2}}{1} \tag{3.19}$$

From equation (3.12) can be easily verified that at node i, the displacement (u_i) onto the local x axis equals the summation of all the projections of D_{3i-2} and D_{3i-1}, and the summation of all the projections of D_{3i-2} and D_{3i-1} onto the local y axis equals to v_1.

The same procedure can be said at node *j*. The matrix T for a frame element transforms a 6 x 6 matrix into another 6 x 6 matrix. Using the transformation matrix, T, the matrices for the frame element in the global coordinate system becomes

$$\boldsymbol{K_e} = \boldsymbol{T^T k_e T} \tag{3.20}$$

$$\boldsymbol{M_e} = \boldsymbol{T^T m_e T} \tag{3.21}$$

$$\boldsymbol{F_e} = \boldsymbol{T^T f_e} \tag{3.22}$$

Note that there is no change in dimension between the matrices and vectors in the local and global coordinate systems.

3.3.2 Space frames

3.3.2.1 Equations in local coordinate system

The 3D frame elements can be taken from the 2D frame elements by developing approach. The only difference is that there are more DOFs at a node in a 3D frame element than in a 2D frame element. There are altogether 6 DOFs in each node in a three-dimensional frame element: three translational displacements and three rotations with respect to the x, y and z axes, and it becomes 12 DOFs for an element with two nodes, as shown in Figure 3.7.

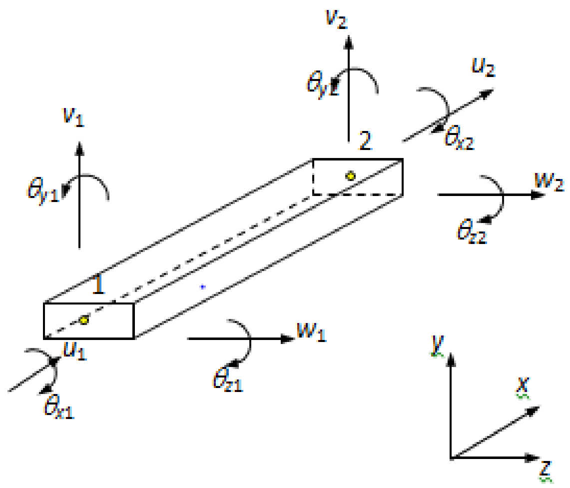


Figure 3.7 Frame elements in space with twelve DOFs.

The element displacement vector for a frame element in space can be written as

$$\mathbf{d}_e = \begin{Bmatrix} d_1 \\ d_2 \\ d_3 \\ d_4 \\ d_5 \\ d_6 \\ d_7 \\ d_8 \\ d_9 \\ d_{10} \\ d_{11} \\ d_{12} \end{Bmatrix} = \begin{Bmatrix} u_1 \\ v_1 \\ w_1 \\ \theta_{x1} \\ \theta_{y1} \\ \theta_{z1} \\ u_2 \\ v_2 \\ w_2 \\ \theta_{x2} \\ \theta_{y2} \\ \theta_{z2} \end{Bmatrix} \begin{matrix} \left.\begin{matrix} \\ \\ \\ \\ \\ \\ \end{matrix}\right\} \text{displacement components at node 1} \\ \left.\begin{matrix} \\ \\ \\ \\ \\ \\ \end{matrix}\right\} \text{displacement components at node 2} \end{matrix} \tag{3.23}$$

Also the element matrices can be obtained from the matrices of the space truss element and that of beam elements, and combining them together by same similar procedure.

$$\mathbf{k}_e = \begin{bmatrix}
\frac{AE}{2a} & 0 & 0 & 0 & 0 & 0 & \frac{-AE}{2a} & 0 & 0 & 0 & 0 & 0 \\
 & \frac{3EI_z}{2a^3} & 0 & 0 & 0 & \frac{3EI_z}{2a^2} & 0 & \frac{-3EI_z}{2a^3} & 0 & 0 & 0 & \frac{3EI_z}{2a^2} \\
 & & \frac{3EI_y}{2a^3} & 0 & -\frac{3EI_y}{2a^2} & 0 & 0 & 0 & \frac{3EI_y}{2a^3} & 0 & \frac{-3EI_y}{2a^2} & 0 \\
 & & & \frac{GJ}{2a} & 0 & 0 & 0 & 0 & 0 & \frac{-GJ}{2a} & 0 & 0 \\
 & sy. & & & \frac{2EI_y}{a} & 0 & 0 & 0 & \frac{3EI_y}{2a^2} & 0 & \frac{EI_y}{a} & 0 \\
 & & & & & \frac{2EI_z}{a} & 0 & \frac{-3EI_z}{2a^2} & 0 & 0 & 0 & \frac{EI_z}{a} \\
 & & & & & & \frac{AE}{2a} & 0 & 0 & 0 & 0 & 0 \\
 & & & & & & & \frac{3EI_z}{2a^3} & 0 & 0 & 0 & -\frac{3EI_z}{2a^2} \\
 & & & & & & & & \frac{3EI_y}{2a^3} & 0 & \frac{3EI_y}{2a^2} & 0 \\
 & & & & & & & & & \frac{GJ}{2a} & 0 & 0 \\
 & & & & & & & & & & \frac{2EI_y}{a} & 0 \\
 & & & & & & & & & & & \frac{2EI_z}{a}
\end{bmatrix} \tag{3.24}$$

Where

I_y and I_z are moment of inertia of the cross section beam, with respect to the y and z axes. The only difference is that the axial deformation is replaced by the torsional angular deformation, and axial force is replaced by torque. It means that , the element tensile stiffness *AE/2a* is replaced by the element torsional stiffness *GJ/2a*, where *G* and J are shear modules and polar moment of inertia of the cross-section of the bar respectively.

The m_e is also shown as follows:

$$
m_e = \frac{\rho A a}{105}\begin{bmatrix}
70 & 0 & 0 & 0 & 0 & 0 & 35 & 0 & 0 & 0 & 0 & 0 \\
 & 78 & 0 & 0 & 0 & 22a & 0 & 27 & 0 & 0 & 0 & -13a \\
 & & 78 & 0 & 13a & -6a^2 & 0 & 0 & 27 & 0 & 13a & 0 \\
 & & & \frac{70 r_x^2}{1} & 0 & 0 & 0 & 0 & 0 & \frac{-35 r_x^2}{1} & 0 & 0 \\
 & \text{sy.} & & & 78 & -22a & 0 & 0 & -13a & 0 & -6a^2 & 0 \\
 & & & & & 8a^2 & 0 & 13a & 0 & 0 & 0 & -6a^2 \\
 & & & & & & 70 & 0 & 0 & 0 & 0 & 0 \\
 & & & & & & & 78 & 0 & 0 & 0 & -22a \\
 & & & & & & & & 78 & 0 & 22a & 0 \\
 & & & & & & & & & \frac{70 r_x^2}{1} & 0 & 0 \\
 & & & & & & & & & & 8a^2 & 0 \\
 & & & & & & & & & & & 8a^2
\end{bmatrix}
$$

Where

$$r_x^2 = \frac{I_X}{A} \tag{3.26}$$

3.3.2.2 Equations in global coordinate system

After preparing the element matrices in the local coordinate system, the next thing to do is transformation of the element frame matrices from the local coordinate into the global coordinate system that are attached on individual frame members.

Assume that the local nodes 1 and 2 of the element correspond to global nodes *i* and *j* respectively.

The displacement at a local node should have three translational components, and three rotational components in the x, y and z directions, and sequentially numbered as d_1-d_{12} corresponding to the physical deformations as defined by equation (3.23). Also the node displacement at a global coordinate should have three translational and three rotational components with respect to global X, Y and Z axes. See Figure 3.8.

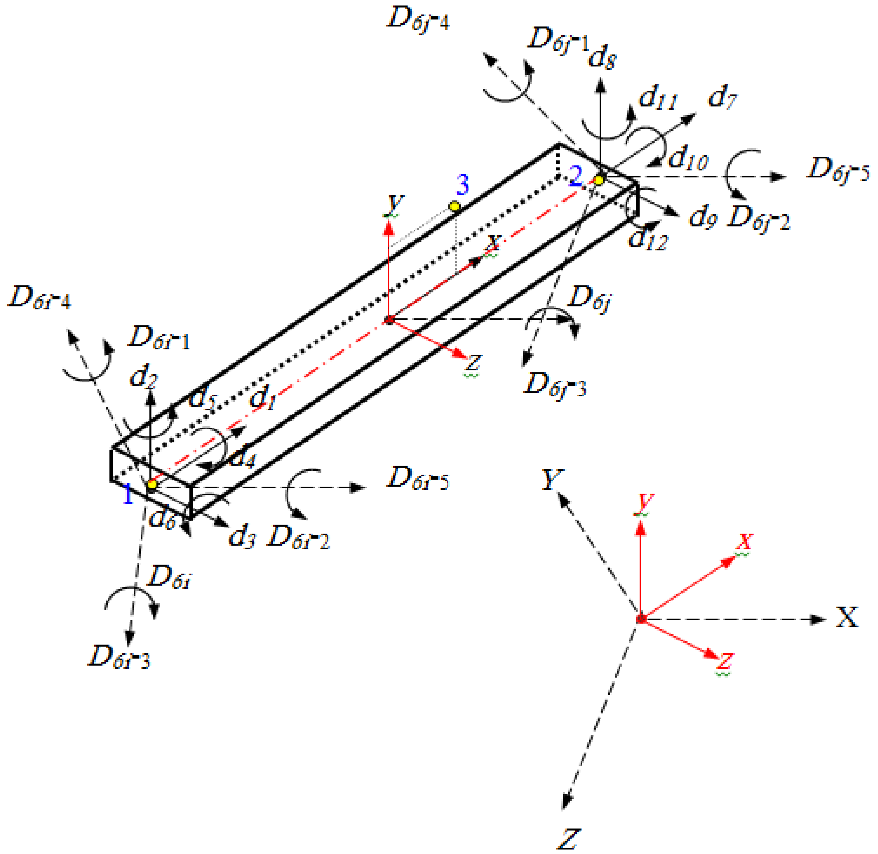


Figure 3.8 Coordinate transformation for a frame element in space.

$$\mathbf{d_e} = \mathbf{TD_e} \tag{3.27}$$

Where

$$\mathbf{D_e} = \begin{bmatrix} D_{6i-5} \\ D_{6i-4} \\ D_{6i-3} \\ D_{6i-2} \\ D_{6i-1} \\ D_{6i} \\ D_{6j-5} \\ D_{6j-4} \\ D_{6j-3} \\ D_{6j-2} \\ D_{6j-1} \\ D_{6j} \end{bmatrix} \quad (3.28)$$

And **T** is a

$$\mathbf{T} = \begin{bmatrix} \mathrm{T}_3 & 0 & 0 & 0 \\ 0 & \mathrm{T}_3 & 0 & 0 \\ 0 & 0 & \mathrm{T}_3 & 0 \\ 0 & 0 & 0 & \mathrm{T}_3 \end{bmatrix} \quad (3.29)$$

In which

$$\mathrm{T}_3 = \begin{bmatrix} l_x & m_x & n_x \\ l_y & m_y & ny \\ l_z & m_z & n_z \end{bmatrix} \quad (3.30)$$

Where *l, m* and *n* with their subscriptions are cosine directions

$$l_x = \cos(\mathrm{x,X}),\ l_y = \cos(\mathrm{y,X}),\ l_z = \cos(\mathrm{z,X})$$

$$m_x = \cos(\mathrm{x,Y}),\ m_y = \cos(\mathrm{y,Y}),\ m_z = \cos(\mathrm{z,Y})$$

$$n_x = \cos(\mathrm{x,Z}),\ n_y = \cos(\mathrm{y,Z}),\ n_z = \cos(\mathrm{z,Z}).$$

3.4 Static Analysis of Moment Resisting Space Frame

To perform the stiffness matrix formulation which is coded in FORTRAN, two examples are considered. Examples include two 3D moment resisting frame with 63 elements and the results of displacement and stresses are compared with the results of used commercial package SAP2000.

3.4.1 Six story space frame

This example consists of 6 story space moment resisting frame with 63 elements and 180 DOF . The structure is subjected to a gravity load of 19.16 kPa on all floor levels and a lateral load of 109 kN applied at each unrestrained node in the front elevation along the y direction as shown in Figure 3.9. With the square shape (30 x 30) cm, cross section for all horizontal and vertical members, the element and joints numbers are shown in Figures 3.10 and 3.11.

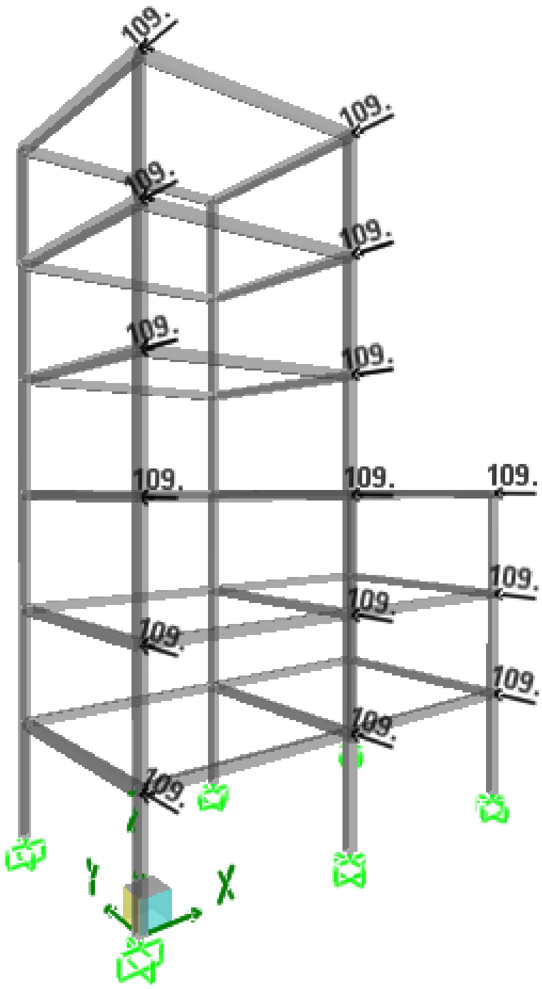


Figure 3.9 Loading of 6 story space frame

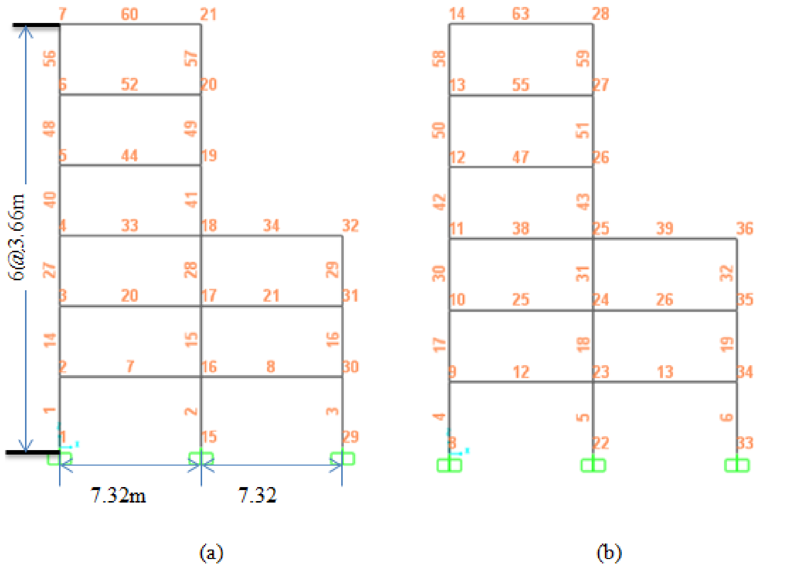


Figure 3.10 Joint and element number in (a) y=0 m and (b) y=7.32 m

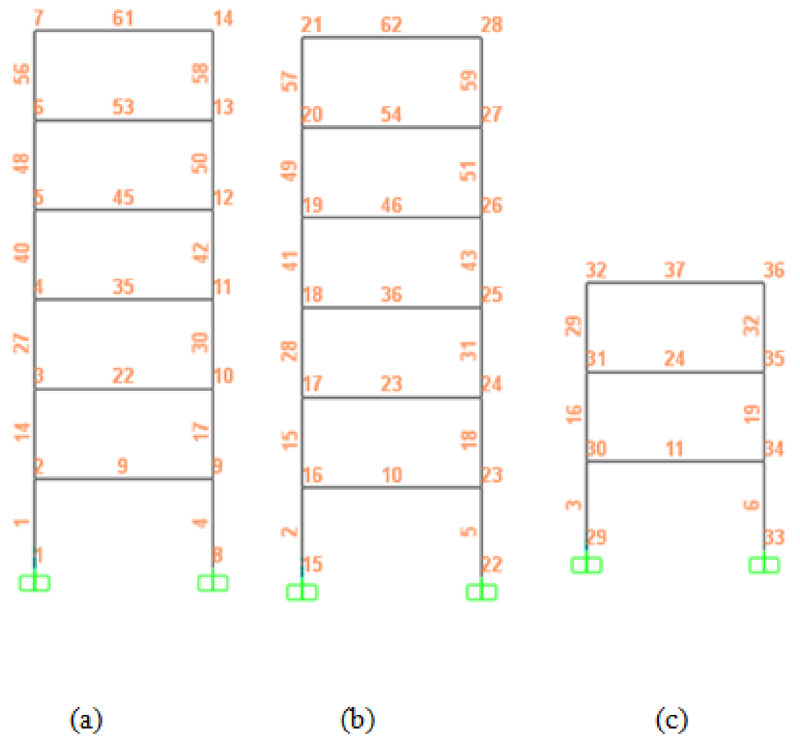


Figure 3.11 Joint and element number (a) x=0 m, (b) x=7.32 m and (c) x=14.64 m

Discussion of the results*:* Analyses are done by the FEM equation which is prepared by FORTRAN code and the results are compared with source programme SAP2000. Table 3.1 shows the result of displacements in x, y and z directions. Maximum displacements occurred in joints (14). Table 3.2 shows the stress of some selected members. Maximum compression stress is occurred in member (4) and maximum tension stress is occurred in member (1)

Table 3.1 Joint displacement (m) of 6 story space frame

Joint no.	x-direction		y-direction		z-direction	
	Present	SAP2000	Present	SAP2000	Present	SAP2000
9	0.001737	0.001771	0.023738	0.023775	-0.000342	-0.000343
10	0.004170	0.004272	0.059624	0.059744	-0.000614	-0.000615
11	0.005869	0.006026	0.091550	0.091743	-0.000815	-0.000816

12	0.006902	0.007072	0.116177	0.116403	-0.000952	-0.000953
13	0.007284	0.007454	0.133155	0.133387	-0.001034	-0.001035
14	0.007448	0.007617	0.142875	0.143106	-0.001071	-0.001071
23	0.001742	0.001777	0.019996	0.019975	-0.000381	-0.000381
24	0.004173	0.004276	0.050102	0.050069	-0.000679	-0.00068
25	0.005871	0.006029	0.077638	0.077624	-0.000896	-0.000897
26	0.006904	0.007073	0.101313	0.101315	-0.001036	-0.001036
27	0.007289	0.007459	0.118431	0.118426	-0.001119	-0.001119
28	0.007435	0.007604	0.128271	0.12826	-0.001156	-0.001156
34	0.001745	0.00178	0.015103	0.015048	-0.000159	-0.000159
35	0.004179	0.004282	0.035949	0.035772	-0.000259	-0.000258
36	0.005864	0.006022	0.050742	0.050415	-0.000304	-0.000302

Table 3.2 Element stress result (kN/m^2) of 6 story space frame

Element no.	SAP2000	Present work
1	209330.03	209253.89
2	-180155.13	-180066.15
3	-133351.6	-133436.16
4	-228078.76	-227897.00
5	-204712.79	-204606.68
6	-155487.06	-155600.98

3.4.2 Three story space frame

This example consists of 3 story space frame with 63 elements Figure 3.12. The structure is divided in to 4 groups and arranged as 1st group is the columns of first and second storeys, 2nd group is the columns of third story, the third group is the all inner beams of all storeys and 4th groups is outer beams of all storeys. The structure is subjected to a gravity load of 12. kPa on all 4th group beams, 16 kPa on all 3rd

group beams, lateral load of 150 kN in direction of wind in x-direction and 80 kN in leeward side. With the (40 x 40) cm, cross section for Group1, (35 x35) cm for second group, (40 x30) cm for third group and (40 x25) cm for fourth group, the element and joints numbers are shown in Figures 3.13 and 3.14 (a, b, c).

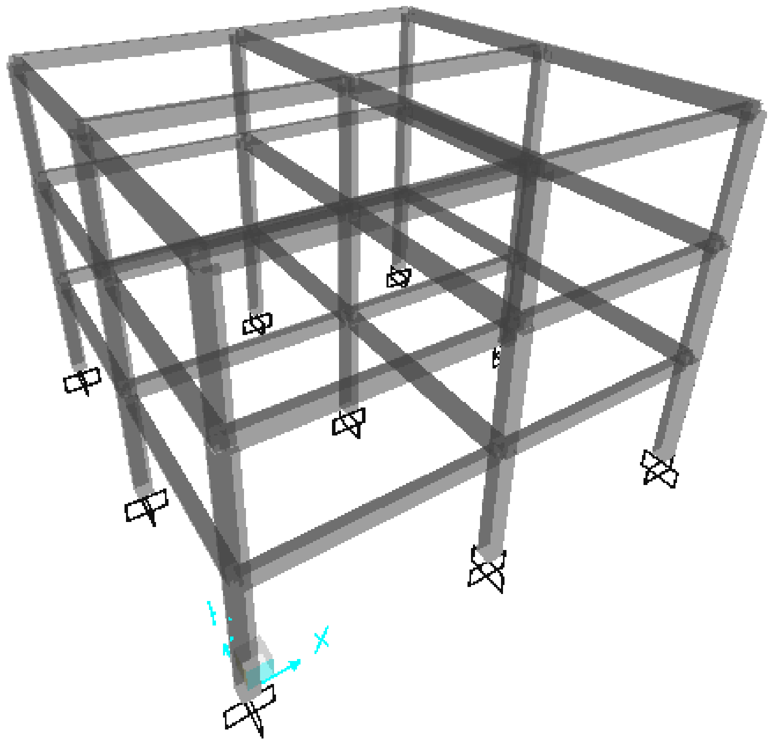
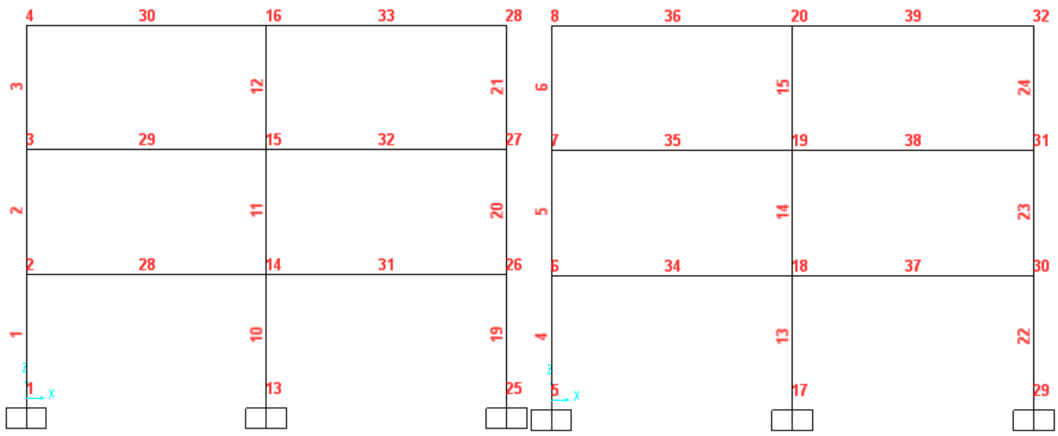


Figure 3.12 3D View of 63 element space frame



(a) (b)

Figure 3.13 Joint and element number of 3 story space frame (a) y= 0 m, (b) y= 6 m

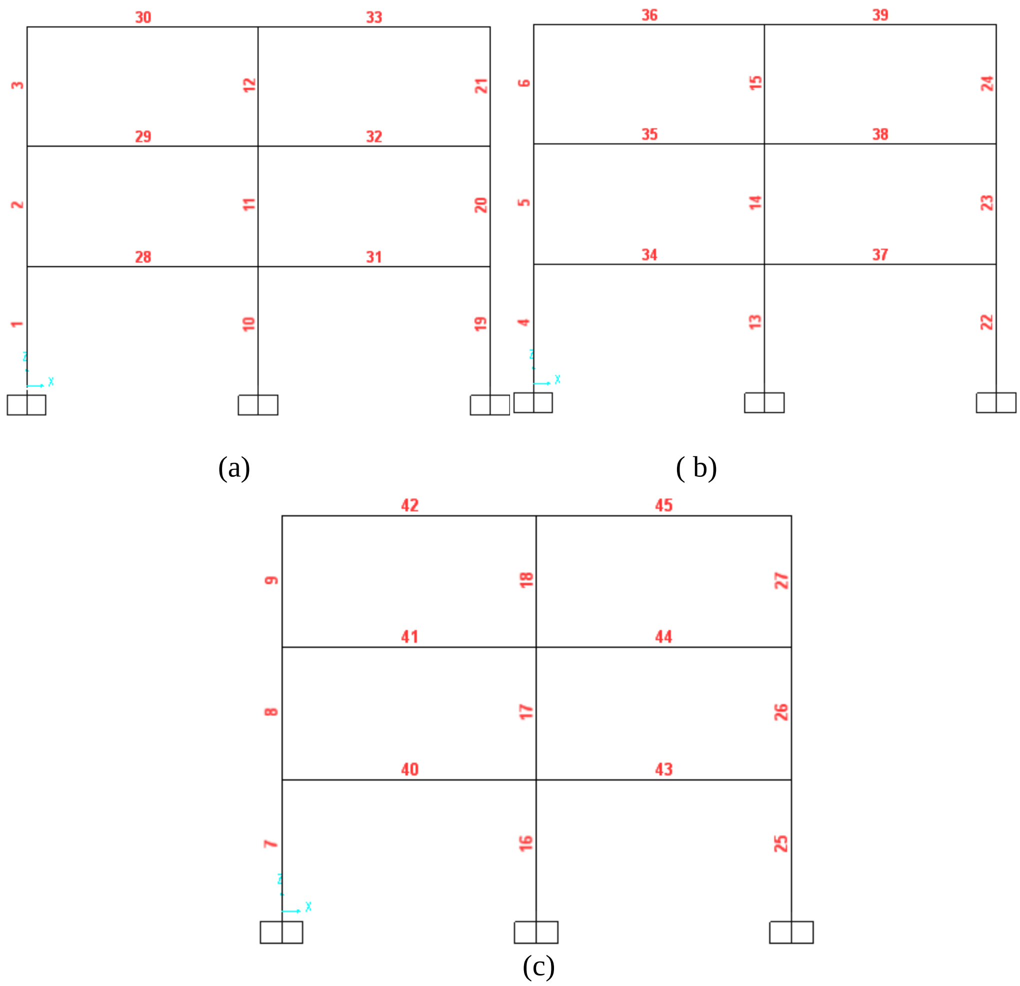


Figure 3.14 Element number of 3 story space frame at (a) y= 0 m, (b) y= 6 m and (c) y=12 m

Discussion of result*:* Analyses are done by the FEM equation which is prepared by FORTRAN code and results are compared with the used source program SAP2000. Maximum displacement occurred in joint 24 as shown in Table 3.3 with the value of 0.011 m. Table 3.4 shows the stress values for some selected vertical elements the maximum tension stress occurred at element 4 with the value of 48306.04 kN/m^2 and maximum compression occurred at element 22 with the value of -56951.03 kN/m^2 at element 22.

Table 3.3 Joint displacement (m) of 3 story space frame

Joint no.	x-direction	
	Present	**SAP2000**
2	0.0031737	0.003208
3	0.0072170	0.007465
4	0.01104	0.010513
6	0.0031737	0.003208
7	0.0072170	0.007465
8	0.01104	0.010513
10	0.0031737	0.003208
11	0.0072170	0.007465
12	0.01104	0.010513
14	0.0031737	0.003193
15	0.0072170	0.00744
16	0.01104	0.010479
18	0.0031737	0.003193
19	0.0072170	0.00744
20	0.01104	0.010479
22	0.0031737	0.003193
23	0.0072170	0.00744
24	0.01134	0.010479

Table 3.4 Element stress (kN/m^2) of 3 story space frame

Element no.	SAP2000	Present work
1	45099.26	45099.26
2	18886.36	18886.36
3	17072.71	17072.71
4	48306.02	48306.00
5	15692.6	15692.6
6	10388.96	10388.96
7	45099.26	45099.26
8	18886.36	18886.36
9	17072.71	17072.71

10	-56951.03	-56952.01
11	-39148.95	-39148.85
12	-38697.33	-38697.13
13	-58735.1	-58734.87
14	-39478.46	-39479.16
15	-33919.08	-33919.88
16	-56951.03	-56952.01
17	-39148.95	-39148.85
18	-38697.33	-38697.13
19	-52416	-52413.04
20	-27353.23	-27352.83
21	-30328.5	-30329.01
22	-56951.03	-56952.03
23	-39148.95	-39148.85
24	-38697.33	-38697.33
25	-52416	-52416.28
26	-27353.23	-27353.13
27	-30328.5	-30328.4

3.5 Conclusions

The results obtained from the program are compared with other analysis programs to prove the ability and accuracy of FEM analysis in engineering field. This comparison helps to improve the written program to give better solutions.

The close agreement between results is seen the developed FE method program is accurate and robust tools for analysis of space frame structures were developed.

CHAPTER 4

STRUCTURAL OPTIMIZATION

4.1 Introduction

Most design applications in civil engineering involve selecting values for a set of design variables that best describe the behaviour and performance of the particular problem while satisfying the requirements and specifications imposed by codes of practice. Mathematically these design variables are discrete for most practical design problems. In optimization problems with discrete design variables, searching for the local or a global optimal solution becomes a difficult task [28].

The relation between structural form, stiffness and strength in discrete structures has been widely recognized by structural engineers and designers. Such rigid structures have higher resistance against deformation and may therefore be considered structurally more efficient.

In structural design weight minimization is a widespread structural design problem, in which the structures are subjected to more realistic different constraints, like displacements and stresses. These design variables are available in the form of continuous or discrete design variables. The use of a continuous design variable in structural optimization procedure will lead to obtain a set of non-available cross sections and any attempt to substitute those values by the nearest available discrete sizes can potentially make the weight is unnecessarily increased or design violated [29].

4.2 Optimization Techniques

In general, optimization techniques used in structural engineering design can be categorized in to three distinct approaches (1) mathematical programming, (2) optimality criteria methods and (3) Heuristic search methods. These methods are presented in the following sections.

4.2.1 Mathematical programming

Mathematical programming can be classified into linear and nonlinear programming.

4.2.1.1 Linear programming

Linear Programming (LP) is a branch of applied mathematics that deals with solving optimization problems of a particular form. LP problems consist of a certain number of variables in the linear form which is to be minimized or maximized subject to a certain number of constraints. To apply linear programming techniques to structural optimization, the relationship between the objective function and the constraints to the design variables have to be linearized [30]. The main property of LP is that the associated constraints and the objective functions are expressed as a linear combination of the design variables [31].

4.2.1.2 Nonlinear programming

In many interesting maximization and minimization problems, the objective function may not be a linear function, or some of the constraints may not be linear constraints. Such an optimization problem is called a Nonlinear Programming problem (NLP). Nonlinear mathematical model is developed for nonlinear unconstrained optimization problems, in the case when linear relation is used to nonlinear model of structure errors are inevitable., as shown in Figures 4.1 and 4.2 respectively, graphically representation for actual optimization and the linear approximated are difference. Where *F(x)* is the objective function with two design variables x_1, x_2 and $G_1(x)$ and $G_2(x)$ are associated constraints.

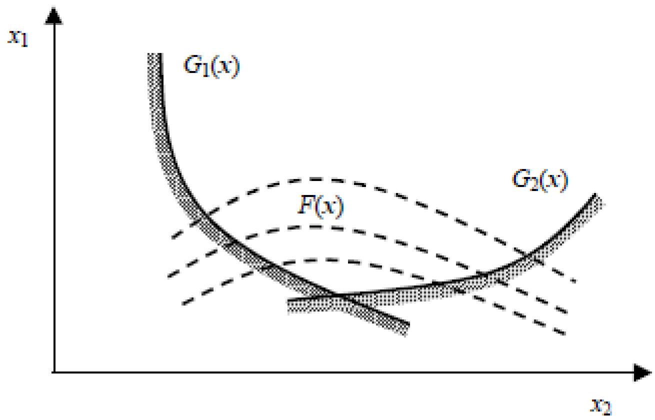


Figure 4.1 Actual optimization problem

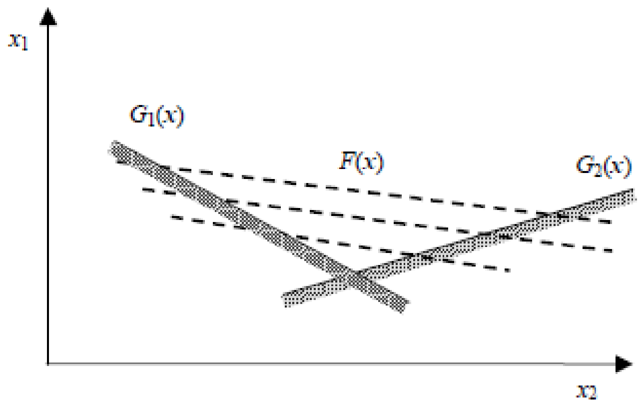


Figure 4.2 Linear approximated problem

4.2.2 Optimality criteria methods

Optimality criteria methods are developed from indirectly applying the Kuhn-Tucker conditions of nonlinear mathematical programming combined with lagrangian multipliers [31]. Typically this method used in case when the number of the constraints is much less than the number of the design variables and design variables are continuous. There are problems associated with the discretization of a continuous solution, the use of a continuous design variable in structural optimization procedure will lead to obtain a set of non-available cross sections and it is difficult to transfer it to discrete one, also any attempt to substitute those values by the nearest available discrete sizes can potentially make the weight is unnecessarily increased or design violated [29]. The matched discrete design variables may result in a frame having a different structural response, which may not satisfy the required performance constraints [31].

4.2.3 Heuristic search methods

A heuristic is defined as an optimization problem technique in which the most appropriate solution of several found is selected at successive stages of a program for further refinement. It provides guidance to reach the most appropriate solution, but in the end is not infallible or fully proven [32].

4.3 Genetic Algorithms

In this study GA is used so detailed explanation is given for this method. There are fascinating algorithms. The name came from the way in which they loosely mimic the process of evolution of organisms, where a problem solution stands in for organism's genetic string. Features include a survival of the fittest mechanism in which potential solutions in a population are pitted against each other, as well as recombination of solutions in mating process and random variations. The incredible part is that this heuristic can "evolve" better and better solutions without any deep understanding of the problem itself.

The introduction of the large-scale digital computers allowed the adaptation of classic optimization algorithms to realistic engineering problems, as well as the advancement of new and more powerful techniques. The investment of time and resources required to develop an optimization capability can be considerable and the projected results must justify the costs.

GAs is the most well-known computational technique based on the principles of evolution. They have been recently introduced in the engineering fields and in architecture as optimization form generation tools, while their algorithmic processes more closely resemble natural evolution than other adaptive search algorithms [33].

GAs can be applied to any problem that has these two characteristics: (i) a solution can be expressed as a string, and (ii) a value representing the worth of the string can be calculated.

4.3.1 GAs versus traditional methods

GA has a couple of important advantages. They are simple to program and they work directly with complete solutions: unlike branch and bound, there is no need for estimates or for bounding function.

GA differs substantially from more traditional search and optimization methods. The most significant differences are:

- A population of points is applied to starting the technique instead of a single design point. Then a several points are applied as elect results, GAs is less probable to catch confined in a local optimum.

- The objective function value only is used by GAs. The mathematical forms were not applied in the search process.

- In GAs the design variables are characterized as a binary string variable which in normal genetic is related to the chromosomes. So the method of search is normally valid to solve integer and discrete examples dealing with programming.

- The string length can be varied to get any preferred solution, for continuous design variables.

- In normal genetics the objective function value relating to a design function plays the role of fitness.

4.3.2 Selection of chromosomes

At this stage the fittest individuals from the present generation are selected. The selection of suitable chromosome is the greatest for the possibility of being chosen for reproduction. Thus, the basis selection is on the survival-of-the-fittest approach with respect to Darwin's theory of evolution, but the main theory in choosing the best individuals in the population.

4.3.3 Problem definition in genetic algorithm

The GA is used to solve the following problem.

$$\text{To minimize} \quad F(\mathbf{s}) \tag{4.1}$$

$$\text{Subjected to} \quad g_j(s) \leq 0 \qquad j = 1,.......,m$$

$$s_i^l < s_i < s_i^u \, , \; i = 1, 2, \ldots, n \qquad (4.2)$$

Where, $\mathbf{s}$ is the vector of design variables, and $F(\mathbf{s})$ is the objective function to be minimized. s_i^l and s_i^u are the lower and upper bounds on a typical design variable s_i , $g_{j(s)}$ are the behavioural constraints.

4.3.4 Parameters used in genetic algorithm

In addition to the main GA operators, there are some parameters. Like chromosomes, string length and population size.

4.3.4.1 Chromosomes

Each chromosome which is represented by the binary alphabet (0, 1) is a legal solution to the problem and is composed of a string of genes. According to application type, chromosomes may be presented as integer or real numbers.

4.3.4.2 String length

The string length represents each design variable and determines the size of the space search, the longer the string length the bigger the search space. In the traditional GA, the strings represents as a fixed length binary string [34].

4.3.4.3 Initial population

Once a suitable representation has been decided upon for the chromosomes, it is necessary to create an initial population to serve as the starting point for the genetic algorithm. A randomly-generated initial population is usually of fairly low quality, the genetic algorithm will do much better if provided with a relatively high quality initial population, but the initial population must also include a certain amount of diversity. The population size depends on the type of encoding and the problem, usually recommended between 30 and 100. Population size says how many chromosomes are in one generation, if there are only few chromosomes, then GA would have a few possibilities to perform crossover and only a small part of search space is explored. Research shows that after some limit, it is not useful to increase population size, because it does not help in solving the problem faster.

4.3.5 Main operators in basic genetic algorithm

The existing GAs are founded upon three main principles namely selection, crossover and mutation. In each iteration, or generation cycle, these operators are applied on a population of possible solutions, or individuals in order to improve their fitness [35].

4.3.5.1 The reproduction operator

The reproduction is equivalent to “survival of the fittest” contest. It determines not only which solutions survive, but how many copies of each of the survivors to make. This will be important later during the crossover operation. Main objective of this process is to allow the stored information in strings with good fitness values to survive into the next generation.

After the reproduction operation, we have an intermediate population known as the mating pool that is ready to mix and mingle, akin to the process of mating and reproducing children that share some of the genetic material of each parent. This is the function of the crossover operator.

Many reproduction operators exist and they all essentially do same thing. They pick from current population the strings of above average and insert their multiple copies in the mating pool in a probabilistic manner.

However, in reproduction process the features of parent strings does not change, the next generation of solution strings are developed from selected pairs of parent's strings and the application of other explorative operators like crossover and mutation [28].

4.3.5.2 Crossover operator (Recombination)

Crossover is a genetic operator that combines (mates) two chromosomes (parents) to produce a new chromosome (offspring), it permits the exchange of genetic material between the two individuals involved producing two new points in the optimization space [36]. The idea behind crossover is that the new chromosome may be better than both of the parents if it takes the best characteristics from each of the parents.

Crossover selects genes from parent chromosomes and creates a new offspring, occurs during evolution according to a user-definable crossover probability.

This happens as follows:

1. The reproduction operator selects randomly a pair of two individual strings for the mating.

2. Randomly select a cross site in the solution string. This is the point between any two positions in the solution string.

3. Swap the ends of the two parent strings, from the crossover point to the end of the string, to create two new child strings.

There are numerous crossover schemes, such as one point crossover, Figure 4.3 two point crossovers, Figure 4.4 uniform and multi point crossover, Figure 4.5 It should be noted that adding further crossover points reduces the performance of the GA [34]. However, an advantage of having more crossover points is that the problem space may be searched completely, but problem in adding an additional crossover points is that building blocks are more likely to be disrupted.

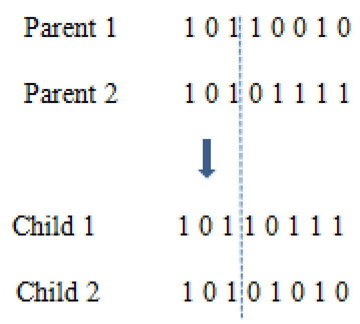


Figure 4.3 Single point crossover

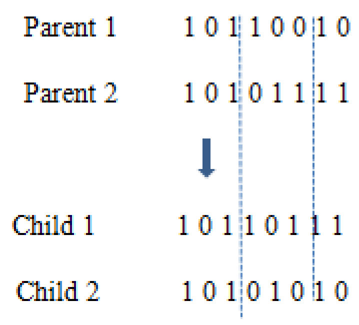


Figure 4.4 Two point crossover

Parent1	1	1	0	1	1	0	0	1	0	0	1	1	0	1	1	0
Parent2	1	1	0	1	1	1	1	0	0	0	0	1	1	1	1	0
Offspring1	1_1	1_2	0_2	1_1	1_1	1_2	1_2	0_2	0_1	0_1	0_2	1_1	1_2	1_1	1_1	0_2
Offspring2	1_2	1_1	0_1	1_2	1_2	0_1	0_1	1_1	0_2	0_2	1_1	1_2	0_1	1_2	1_2	0_1

Figure 4.5 Uniform crossover

4.3.5.3 The mutation operator

The mutation is a secondary GA operator. The mutation operator is used to randomly alter the values of some of the positions in some of the strings based on a parameter that determines the level of mutation. One common choice is a 1 in 1000 chance of mutation. This can be implemented as follows. For each position in each string, generate a random integer between 1 and 1000. If this number is 1, then the position is chosen for mutation, and is randomly switched to any other possible value.

Mutation is a vital part of the solution process, and the mutation rate can have a big impact on the quality of the final solution. It is even possible (though vastly more inefficient) to solve problems using only the mutation operator.

4.3.6 Overview of the fundamental GA operation

At present, that we have seen the fundamental GA operators, we can place the full process with each other. In this place are the necessary steps:

1. Design the algorithm: select the population magnitude n as well as mutation rate; select the operators with the stopping surroundings (further on stopping conditions later).

2. Randomly generate an initial population (further on generating the initial population later) also estimate the fitness value for each string. Set the solution through the best value of the fitness function in the initial population.

3. Apply the reproduction operator to the current population to generate a mating pool of size **n**

4. Apply the crossover operator to the strings in the mating pool to generate a tentative new population of size **n**.

5. Apply the mutation operator to the experimental new population to produce the last new population. Compute the fitness values of the solution strings in the new population also renew the incumbent solution if there is a best solution in this population.

6. If the stopping conditions are met, then exit with the incumbent solution like the last solution. In other, respects go to Step 3.

4.3.7 Stopping conditions

At this stage, the individuals resulting from the process of crossover and mutation are inserted into the new population [36].

The convergence can be done in several ways, depending on the problem. The most obvious way is simply to stop when the specified number of generation's has evolved. But perhaps it would be better to stop when there is very little change between generations, indicating that the evolutionary process has reached a plateau. The genetic process will end if there is no change to the population's best fitness for a specified number of generations, but it is not a good idea to stop, since this does not really measure the amount of ferment going on in the current population. To capture this, the genetic algorithm is sometimes stopped when the average population solution value has not changed for several generations. However even this measure does not always represent the amount of change going on in the current population. This is perhaps better represented by a surprising measure: stop when the worst solution string fitness in the population has not changed for several generations. It is the worst solution value that usually changes the most between generations; when it settles down it is usually true that the whole population has settled down so that more useful new solutions are unlikely to arise.

4.4 Optimization by Using of SAP2000

Most of available commercially structural analysis programs are not designed for optimization but it will be able to achieve this task with preparing open optimization designed code and interfacing them together [2].

SAP2000 could export or import analysis and design data with extension data base file, Microsoft excel text file and Microsoft access. The interaction with it occurs through the input (*$2k) and output files (*out).

SAP2000 structure analysis program is a well-known integrated FE structural analysis tool which already used for analysing, modelling and design of structures according to different design codes. OAPI in SAP2000 is a programming tool which aims to offer efficient access to the analysis and design technology of the SAP2000 structural analysis software, by allowing, during run-time, a direct bind to be established, between a third-party application and the analysis software itself Figure 4.6. Additionally, through the use of this OAPI, one has the option of developing plug-ins, which extend the program's usability and are totally embedded within the SAP2000 environment. In terms of computer programming, the OAPI consists of a software library that offers access to a collection of objects and functions capable of "remotely" controlling the way that the SAP2000 behaves, thus, overriding the standard point-and-click procedure.

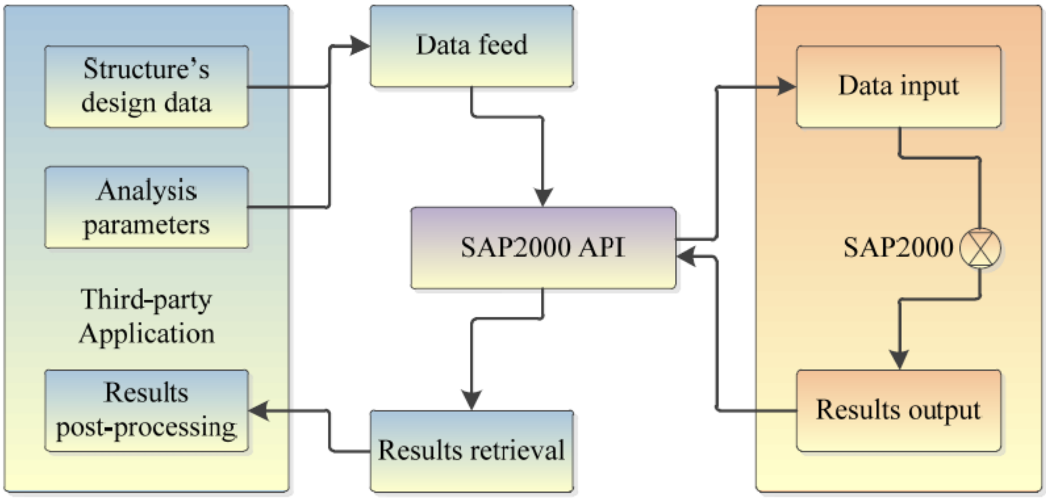


Figure 4.6 Interfacing and typical data flow using the SAP2000 OAPI

4.5 Constraints in Structural Optimization

The introduction of the large-scale digital computers allowed the adaptation of classic optimization algorithms to realistic engineering problems, as well as the advancement of new and more powerful techniques to obtain the optimum design of structural systems. Most of them deal continuous design variables with simple constraints. Only a few of these papers deal with the discrete design variables and actual design constraints according to different structural design code [37, 38, and 39], most of them used optimality criteria methods and mathematical programming techniques with continuous design variables as an optimization tool [18].

The optimization algorithms of steel structures subjected to the actual constraints is done. These constraints are stress and displacement.

The AISC-LRFD specification combines strength, stability and displacement requirements. Displacement constraints are the allowable interstory drift. These constraints are implicit constraints because structural responses like stresses, strains, and displacements are functions of design variables [17].

4.5.1 Stress constraints

Traditionally, structural steel design has been based on allowable stress design ASD, in ASD allowable stress of a material is compared to calculated working stress resulting from service loads. In 1986, AISC introduced a specification based entirely on LRFD for design of structures. In 2005, AISC introduced a unified specification in which both methods were incorporated, both based on the nominal strength of a member, and this principle is continued in the 2010 Specification [40].

4.5.1.1 Stress constraints according to LRFD

According to (AISC-LRFD) specification the allowable stress for members subject to bending and axial force are [41].

$$\text{For} \quad \frac{P_u}{\varphi\, P_n} \geq 0.2 \tag{4.3}$$

$$\frac{P_u}{\phi P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0 \tag{4.4}$$

$$\text{For} \quad \frac{P_u}{\varphi P_n} < 0.2 \tag{4.5}$$

$$\frac{P_u}{2\phi P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \leq 1.0 \quad (4.6)$$

In equation 4.4 and 4.6 if the axial force is in compression or in tension the terms in above equations are

P_u is the required axial strength (tension or compression); P_n nominal axial strength (tension or compression), M_{ux} is the required flexural strength about the minor axis, M_{uy} is the required flexural strength about the minor axis, M_{nx} is the nominal flexural strength about the major axis, M_{ny} is nominal flexural strength about the minor axis,

(for 2D structures, M_{uy} is equal to zero); $\phi = \phi_t$ resistance factor for tension (equal to 0.90)

$\phi = \phi_c$ compression resistance factor and ϕ_b = flexural resistance reduction factor =0.9.

The nominal compressive strength of a member is computed as

$$P_n = A_g . F_{cr} \quad (4.7)$$

$$F_{cr} = \left(0.658 \, \lambda c^2 \right) . F_y \quad (4.8)$$

Where $\lambda c \leq 1.5$

$$F_{cr} = \left(\frac{0.877}{\lambda c^2} \right) . F_y \quad (4.9)$$

For $\lambda c > 1.5$

$$\lambda_c = \frac{KL}{r\pi} \sqrt{\frac{F_y}{E}} \quad (4.10)$$

In which

A_g is the gross cross-sectional area of a member, K = Effective length factor for braced and un braced member [33].

E =modulus of elasticity of a member, r = radius of gyration, L = length of member, and F_y = yield stress of steel.

4.5.1.2 Stress constraints according to AISC – allowable stress design

The members subjected to combined (axial compression and flexural stress constraints) taken from AISC (1989) must be sized to meet the following constraints [42].

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} A_1 + \frac{f_{by}}{F_{by}} A_2 \leq 1.0 \tag{4.11}$$

$$\frac{f_a}{0.6F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \tag{4.12}$$

$$A_1 = \frac{C_{mx}}{\left(1 - \frac{f_a}{F_{EX}}\right)} \quad , \quad A_2 = \frac{C_{my}}{\left(1 - \frac{f_a}{F_{EY}}\right)} .$$

When $\frac{f_a}{F_a} \prec 0.15 \quad \rightarrow \quad A_1 = A_2 = 1.0$

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \tag{4.13}$$

In the equations (4.11 to 4.13) the subscripts x and y, combined with subscripts b and m indicate the axis of bending about which a particular stress or design property applies.

F_a = allowable axial stress under axial compression force only, F_b = allowable bending compressive stress that would be permitted if bending moment alone existed,

F_{EX} , F_{Ey} is the Euler stress divided by a factor of safety of 23/12, $f_a = (P/A)$ represents computed axial stress, f_b is the computed flexural bending stress at the point under consideration, and C_m = a coefficient whose value is taken as 0.85 for compression members in frames subject to sideway [18].

For members subjected to both axial tension and bending stresses,

$$\frac{f_{ta}}{F_t} + \frac{f_{btx}}{F_{btx}} + \frac{f_{bty}}{F_{bty}} \leq 1.0 \tag{4.14}$$

In equation (4.14), f_{bt} = computed bending tensile stress, f_{ta} is the computed axial tensile stress, F_{bt} is the allowable bending stress and F_t is the governing allowable tensile stress.

4.5.2 Serviceability limit states (displacement constraints)

The increasing use and reliance on probability based limit states design methods, such as the recently adopted AISC- LRFD Specification [41], has concentrated new attention on the serviceability problems in steel buildings. These methods, along with the development of higher-strength building materials and the use of lighter and less rigid building materials, have led to more flexible and lightly damped structures than ever before, making serviceability problems more prevalent.

Lateral frame movement or deflection is usually evaluated for the building as a whole, where the applicable parameter is total building drift, which is equal to (D/ H) where D is total top-story drift and H is the total structure height, and for each floor of building which is known as inter-story drift, and can be defined as the lateral deflection of a floor relative to the one immediately below it divided by the distance between floors ((d*n*–(d*n*–1)/ *h*) [43]).

Where *dn* is the drift of specified floor and h is equal to height of that floor.

According to the ASCE report [44], normally allowable accepted ranges for lateral displacements are restricted between 1/750–1/250 times the building heights with a typical value of H/400 and the normally accepted limits on the inter-story drift is 1/500–1/200 times the story height with a typical value h/300; where h is the height of an story.

Based on Ellingwood [45] the deflection limits for story are selected as:

Lateral allowable drift is H/400 for the case of service wind load and H/300 for interstory drift.

4.5.3 Fabrication constraints

The fabrication constraint is that, structure elements are available in the form of discrete sections; otherwise the algorithm would not have any practical application.

The available steel frame sections do not exist on continuous domains based on cross-sectional area or strong-axis moment of inertia, structural optimization problems are most commonly formulated on discrete variable spaces. In discrete formulations, the design variables are not continuous. The standard available steel sections are treated as design variables and the stress and displacement constraints are taken from the design codes [46]. Traditionally, in design of space moment frames, frame members (column and beams) are usually selected W-sections, so a file with different section property is prepared, for beams and columns. For example, consider a framed structure, where the structure is subjected to design stress, displacement and fabrication constraints the equation of optimization problem may be expressed as

$$\text{Minimize} \qquad W = \sum_{i=1}^{N} \rho_i l i A_i (\eta_i) \tag{4.15}$$

$$\text{Subjected to} \qquad (\sigma^u) \geq (\sigma) \geq (\sigma^l) \tag{4.16}$$

$$(d^u) \geq (d) \geq (d^l) \tag{4.17}$$

$$(A^u) \geq (A) \geq (A^l) \tag{4.18}$$

Where σ , d , A are stress, displacement and cross sectional area and subscripts *u* and *l* refer to prescribed upper and lower boundaries of each constraints.

η_i Index number according to fabrication code

$A_i(\eta_i)$ Is the cross sectional area of element *i*.

CHAPTER 5

NUMERICAL OPTIMIZATION

5.1 Space Frame Examples

In this part 3D moment frame examples are optimized under static loads. The objective function is weight minimization under stress and displacement constraints according to AISC-LRFD specifications, with the 4 load combinations as shown in Table 5.1.

Table 5.1 Load combination case

Load case	Combinations
Comb1	$1.4D_l$
Comb2	$1.2D_l+1.6L_l+0.5R_l$
Comb3	$1.2D_l+0.5L_l+1.6R_l$
Comb4	$1.2D_l+1.3\ W_l+0.5L_l+0.5R_l$

5.1.1 One bay space moment frame

This example is consist of 8 member space moment frame as shown in the Figure 5.1, the structure subjected to Live load (Ll= 2.39 kPa) , Dead load (Dl=2.78 kPa), Roof live load (R_l= 2.39) kPa and Wind pressure (p) = $C_eC_qq_sI$. Where p is design wind pressure; C_e is combined height, exposure and gust factor coefficient, C_q is pressure coefficient is equal to 0.8 and 0.5 for both windward and leeward faces of the structure respectively, q_s is wind stagnation pressure is equal to 0.785 kPa, and the importance factor I=1. The structure is optimized according to AISC- LRFD

Specifications, with maximum drift ratio= 0.004 H, where H= total height of structure. The members of the frame are divided into three groups as shown in Figure 5.1. Joint and element numbers as shown in Figure 5.2 (a) and 5.2 (b) wind load act in the x-direction at each unrestrained node. Material properties for the frame are: Young's modulus $E = 200 \times 10^6$ kN/m^2, material density $\rho = 78.5$ kN/m^3, yield stress fy=344.8 MPa and modulus of rigidity G=83 GPa.

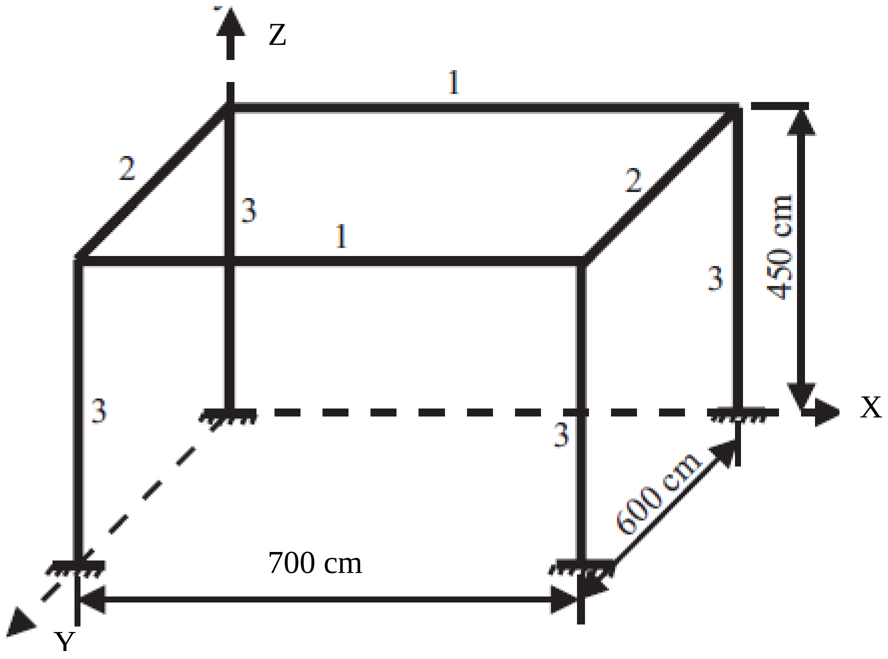


Figure 5.1 One bay space moment frame

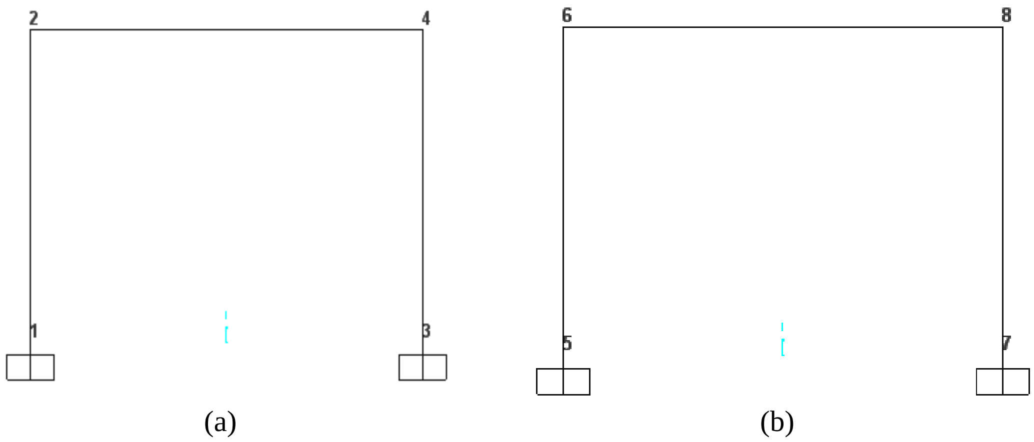


Figure 5.2 Joint number of one bay space moment frame (a) x= 0 and (b) x= 7m

Discussion of results: In Table 5.2 the optimal values of group design variables and weight for the loading combination is considered. The frame was optimized previously by Tabu search [46] the results are compared with other references. maximum allowable drift is 0.004 H= ±1.8 cm, as shown in the Table 5.3 maximum drift for optimized frame is 0.396 cm which less than the allowable top story drift

Table 5.2 Design variables and weight reduction result

Group no.	Design variables	
	Degertekin [46]	**Present work**
1	W16×31	W14×30
2	W16×26	W14×26
3	W8×24	W6×15
Weight(kg)	1678	1425.5

Table 5.3 Joint displacements (m) for all load combinations

Joint No.	Load Combination	X-direction-	Y-direction	Z-direction
2	Comb1	0.0096	0.0025	-0.3510
2	Comb2	0.0190	0.0047	-0.6710
2	Comb3	0.0190	0.0047	-0.6710
2	Comb4	3.9880	0.0045	-0.4710
4	Comb1	0.0096	-0.0025	-0.3510
4	Comb2	0.0190	-0.0047	-0.6710
4	Comb3	0.0190	-0.0047	-0.6710
4	Comb4	3.9880	-0.0045	-0.4710
6	Comb1	-0.0096	0.0025	-0.3510
6	Comb2	-0.0190	0.0047	-0.6710
6	Comb3	-0.0190	0.0047	-0.6710
6	Comb4	3.9610	0.0022	-0.4830
8	Comb1	-0.0096	-0.0025	-0.3510
8	Comb2	-0.0190	-0.0047	-0.6710
8	Comb3	-0.0190	-0.0047	-0.6710
8	Comb4	3.9610	-0.0022	-0.4830

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5.1.2 Four story space moment frame

This example deals with four-story space moment frame shown in Figure 5.3, which is optimized previously by Tabu-search (TS) [46] and Simulated Annealing (AS) [47], the frame members are divided in to 10 groups. The groups were organized as follows: 1-st group: outer beams of 4-th storey, 2-nd group: outer beams of 3-rd, 2-nd and 1-st storeys, 3-rd group: inner beams of 4-th storey, 4-th group: inner beams of 3-rd,

2-nd and 1-st storeys, 5-th group: corner columns of 4-th storey, 6-st group: corner columns of 3-rd, 2-nd and 1-st storeys, 7-th group: outer columns of 4-th storey, 8-th group: outer columns of 3-rd, 2-nd and 1-st storeys, 9-th group: inner columns of 4-th storey, 10-th group: inner columns of 3-rd, 2-nd and 1-st storeys. The height and span lengths of the structure are as shown in Figure 5.4 and 5.5.

The structure subjected to same design loads of example 2 and wind loads on the x-direction for both wind ward and leeward sides. The example is optimized under LRFD stress constraints and 4.55 cm and 1.52 cm for top and inter-storey drift constraints respectively. Material properties for the frame are: Young’s modulus $E = 200 \times 10^6$ kN/m^2, material density $\rho = 7850$ kg/m^3, minimum yield stress fy=248.2 MPa and modulus of rigidity G=83 GPa.

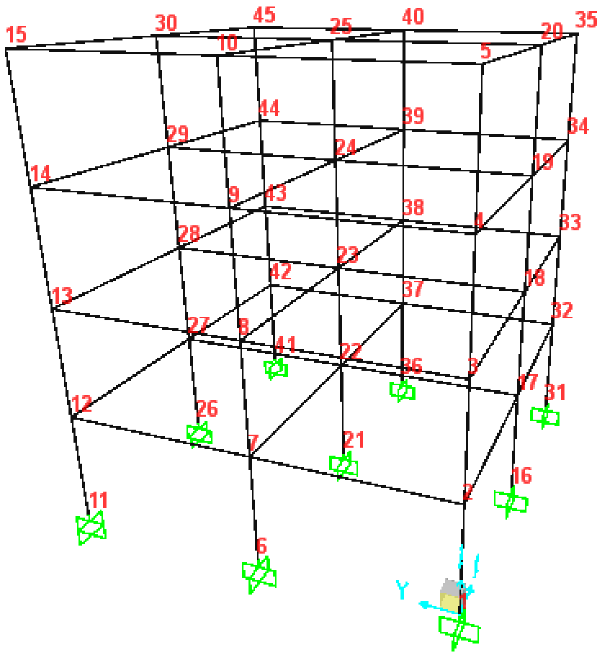

Figure 5.3 Four story space frame with joint numbers

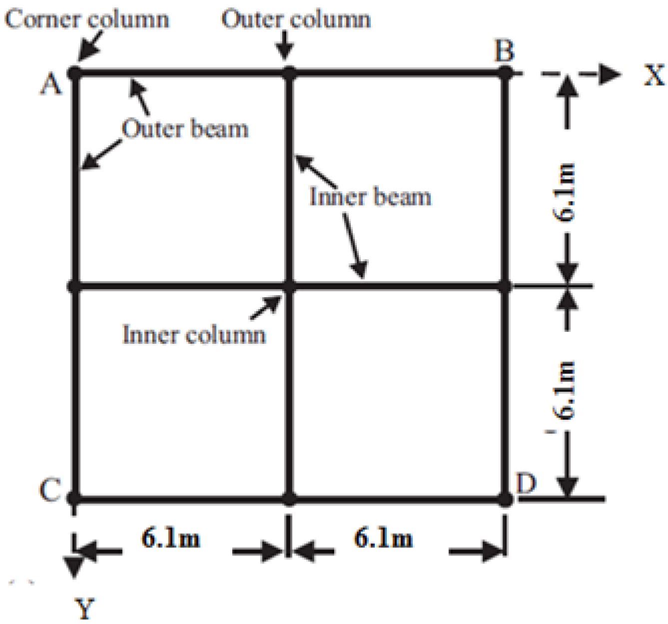

Figure 5.4 Beam of four story moment frame

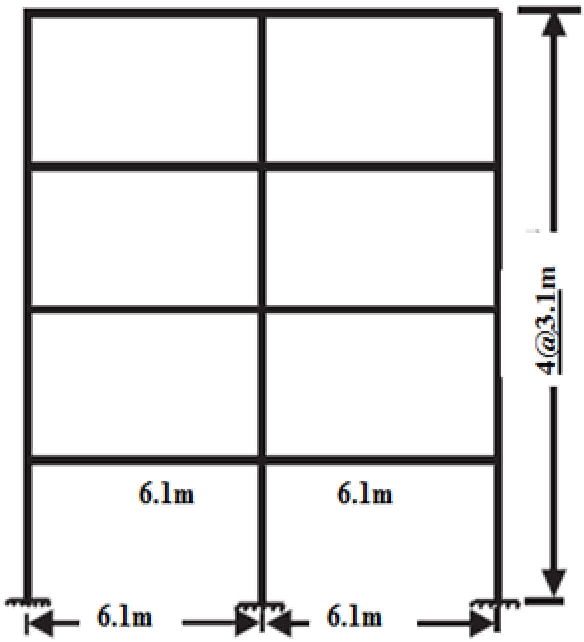

Figure 5.5 Side view, of four story space frame

Discussion of results*:* The 84 element four story moment frames are optimized under AISC-LRFD specification for 4 different load combination cases. The result is compared with the other references [46 and 47] and shown in Table 5.5 which is more close to other references.

The maximum inter-story drift occurred between joints 39 and 40 is equal to 0.22 cm which is less than maximum allowable drift =1.52 cm, and maximum total story drifts occurred in joint 40 =1.4 cm see Table 5.6 which is less than the maximum allowable top-story drift = 5.54 cm as recommended by reference [46] and [47].

Table 5.5 Optimum design variable of four story space frame

Group no.	Design variables		
	Degertekin [47]	Degertekin et al [46]	Present Work
1	W 16×31	W 18×35	W 12×26
2	W 16×31	W 18×35	W 16×36
3	W 18×40	W 18×35	W 18×76
4	W 18×35	W 18×35	W 18×35
5	W 8×35	W 8×31	W 12×30
6	W 14×53	W 12×40	W 16×26
7	W 8×31	W 10×39	W 12×53
8	W 8×35	W 12×45	W 14×43
9	W 8×31	W 8×28	W 6×20
10	W 14×68	W 12×58	W 14×61
Weight(kg)	22405	23105	22961.2

Table 5.6 Displacement (mm) of four story space frame for all load combinations

Joint No.	Load Combination	X-direction-	Y-direction	Z-direction
4	Comb1	-0.0150	-0.0043	-1.3070
4	Comb2	-0.0290	-0.0083	-2.4340
4	Comb3	-0.0290	-0.0083	-2.4340
4	Comb4	8.6650	-0.0032	-1.6270
5	Comb1	0.0490	0.0140	-1.4290
5	Comb2	0.0930	0.0260	-2.6640
5	Comb3	0.0930	0.0260	-2.6640
5	Comb4	10.3450	0.0240	-1.7870
9	Comb1	-0.0160	0.0000	-1.9000
9	Comb2	-0.0280	0.0000	-3.5810
9	Comb3	-0.0280	0.0000	-3.5810
9	Comb4	12.0670	0.0000	-2.4270
10	Comb1	0.0290	0.0000	-2.0690
10	Comb2	0.0570	0.0000	-3.9000
10	Comb3	0.0570	0.0000	-3.9000
10	Comb4	14.4190	0.0000	-2.6510
14	Comb1	-0.0150	0.0043	-1.3070
14	Comb2	-0.0290	0.0083	-2.4340

14	Comb3	-0.0290	0.0083	-2.4340
14	Comb4	8.6650	0.0032	-1.6270
15	Comb1	0.0490	-0.0140	-1.4290
15	Comb2	0.0930	-0.0260	-2.6640
15	Comb3	0.0930	-0.0260	-2.6640
15	Comb4	10.3450	-0.0240	-1.7870
34	Comb1	0.0150	-0.0043	-1.3070
34	Comb2	0.0290	-0.0083	-2.4340
34	Comb3	0.0290	-0.0083	-2.4340
34	Comb4	8.6990	-0.0075	-1.8650
35	Comb1	-0.0490	0.0140	-1.4290
35	Comb2	-0.0930	0.0260	-2.6640
35	Comb3	-0.0930	0.0260	-2.6640
35	Comb4	10.2030	0.0150	-2.0330
39	Comb1	0.0160	0.0000	-1.9000
39	Comb2	0.0280	0.0000	-3.5810
39	Comb3	0.0280	0.0000	-3.5810
39	Comb4	12.0890	0.0000	-2.6890
40	Comb1	-0.0290	0.0000	-2.0690
40	Comb2	-0.0570	0.0000	-3.9000
40	Comb3	-0.0570	0.0000	-3.9000
40	Comb4	14.3300	0.0000	-2.9210
44	Comb1	0.0150	0.0043	-1.3070
44	Comb2	0.0290	0.0083	-2.4340
44	Comb3	0.0290	0.0083	-2.4340
44	Comb4	8.6990	0.0075	-1.8650
45	Comb1	-0.0490	-0.0140	-1.4290
45	Comb2	-0.0930	-0.0260	-2.6640
45	Comb3	-0.0930	-0.0260	-2.6640
45	Comb4	10.2030	-0.0150	-2.0330

5.1.3 10 Story space moment frame

This example deals with 10-storey space moment frame Figure 5.6 with rectangular plane as shown in Figure 5.7. The structure is divided in to 9 groups. The groups are organized as follows: 1-st group: outer beams of top storey, 2-nd group: inner beam of top storey, 3-rd group: outer beams of storeys from 1 to 9, 4-th group: inner beams of storeys from 1 to 9, 5-th group: outer and corner columns of 10-th and 9-th storeys, 6-th group: outer and corner columns of 8-th and 7-th storeys, 7-th group: outer and corner columns of 6-th and 5-th storeys, 8-th group: outer and corner columns of 4-th and 3-rd storeys, 9-th group: outer and corner columns of 2-nd and

1-st storeys.. The structure is subjected to the same design loads and load combinations. The value of qs is 0.622 kPa with wind load in x-direction, the frame joints are as shown in Figure 5.8. The maximum allowable drift is restricted to 18.7 cm.

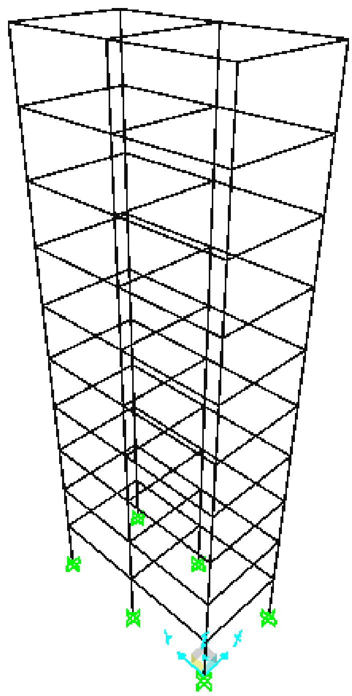

Figure 5.6 10 Story space moment frame

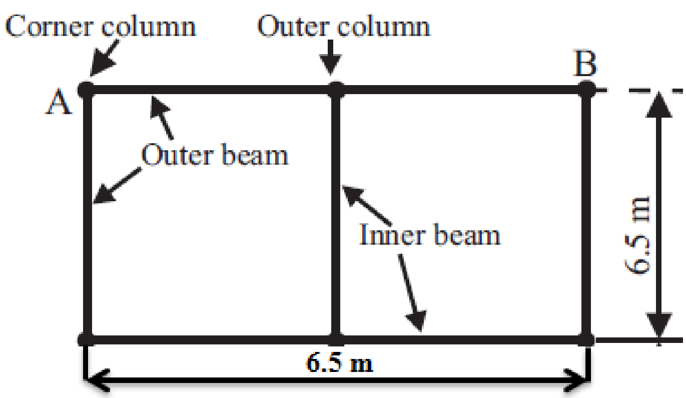

Figure 5.7 Plane of 10 story space moment frame

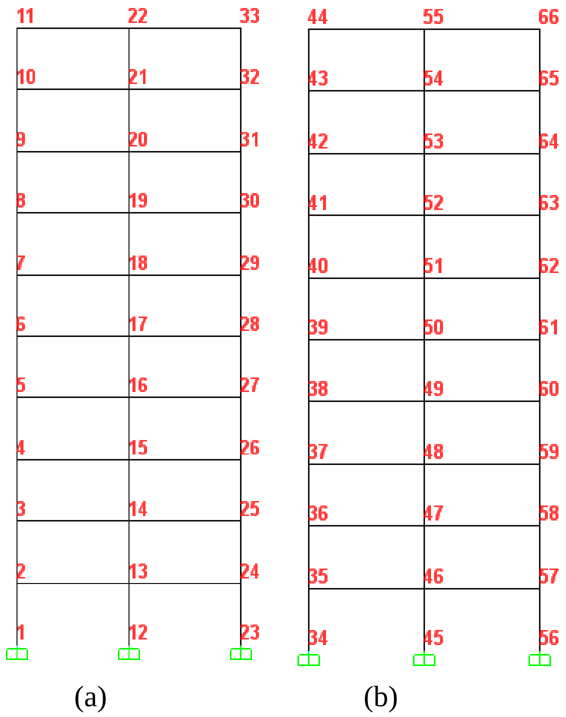

Figure 5.8 Joint number of 10 story space moment frame (a) x= 0 and (b) x= 6.5 m

Discussion of results: The 130 element moment frames are optimized under AISC-LRFD specification for 4 different load combination cases. A minimum weight of 39970 kg is found. The results are compared with the other references [18] as shown in table 5.7 which is more close to them. The maximum displacement occurred in joint 66 which is equal to 15.5 cm is less than the maximum allowable top story drift = 18.7 cm. as shown in table 5.8

Table 5.7 Optimum design variable of 10 story space moment frame

Group no.	**Design variables**	
	Hayalioglu [18]	**Present Work**
1	W14×26	W 14×30
2	W12×40	W 14×34
3	W12×35	W 14×34

4	W12×35	W 14×38
5	W10×22	W 14×53
6	W12×35	W14×48
7	W14×68	W 12×53
8	W14×68	W 12×53
9	W14×82	W14×74
Max. Displacement (cm)	18.1	15.5
Weight(kg)	40976.3	39970

Table 5.8 Joint displacement (mm) for last three storeys

Joint No.	Load Combination	X-direction-	Y-direction	Z-direction
40	Comb1	0.0009	0.0110	-3.3410
40	Comb2	0.0018	0.0220	-6.4810
40	Comb3	0.0018	0.0220	-6.4810
40	Comb4	115.8760	0.0055	-6.0730
41	Comb1	-0.0005	-0.0023	-3.7100
41	Comb2	-0.0011	-0.0047	-7.2000
41	Comb3	-0.0011	-0.0047	-7.2000
41	Comb4	130.1010	-0.0140	-6.6760
42	Comb1	-0.0009	-0.0021	-3.9870
42	Comb2	-0.0019	-0.0044	-7.7390
42	Comb3	-0.0019	-0.0044	-7.7390
42	Comb4	141.3150	-0.0140	-7.1130
43	Comb1	0.0096	-0.0058	-4.1510
43	Comb2	0.0200	-0.0120	-8.0620
43	Comb3	0.0200	-0.0120	-8.0620
43	Comb4	149.4650	-0.0200	-7.3660
44	Comb1	-0.0280	0.0380	-4.2300
44	Comb2	-0.0580	0.0770	-8.2180
44	Comb3	-0.0580	0.0770	-8.2180
44	Comb4	155.0430	0.0430	-7.4840
51	Comb1	0.0016	0.0000	-6.4300
51	Comb2	0.0032	0.0000	-12.8760
51	Comb3	0.0032	0.0000	-12.8760

51	Comb4	139.0260	0.0000	-10.8320
52	Comb1	-0.0009	0.0000	-7.1480
52	Comb2	-0.0018	0.0000	-14.3170
52	Comb3	-0.0018	0.0000	-14.3170
52	Comb4	154.8430	0.0000	-11.9500
53	Comb1	-0.0015	0.0000	-7.6850
53	Comb2	-0.0032	0.0000	-15.3960
53	Comb3	-0.0032	0.0000	-15.3960
53	Comb4	167.2970	0.0000	-12.7690
54	Comb1	0.0160	0.0000	-8.0090
54	Comb2	0.0340	0.0000	-16.0480
54	Comb3	0.0340	0.0000	-16.0480
54	Comb4	176.4090	0.0000	-13.2520
55	Comb1	-0.0470	0.0000	-8.1710
55	Comb2	-0.0990	0.0000	-16.3790
55	Comb3	-0.0990	0.0000	-16.3790
55	Comb4	182.6600	0.0000	-13.4920
62	Comb1	0.0009	-0.0110	-3.3410
62	Comb2	0.0018	-0.0220	-6.4810
62	Comb3	0.0018	-0.0220	-6.4810
62	Comb4	115.8760	-0.0055	-6.0730
63	Comb1	-0.0005	0.0023	-3.7100
63	Comb2	-0.0011	0.0047	-7.2000
63	Comb3	-0.0011	0.0047	-7.2000
63	Comb4	130.1010	0.0140	-6.6760
64	Comb1	-0.0009	0.0021	-3.9870
64	Comb2	-0.0019	0.0044	-7.7390
64	Comb3	-0.0019	0.0044	-7.7390
64	Comb4	141.3150	0.0140	-7.1130
65	Comb1	0.0096	0.0058	-4.1510
65	Comb2	0.0200	0.0120	-8.0620
65	Comb3	0.0200	0.0120	-8.0620
65	Comb4	149.4650	0.0200	-7.3660
66	Comb1	-0.0280	-0.0380	-4.2300
66	Comb2	-0.0580	-0.0770	-8.2180
66	Comb3	-0.0580	-0.0770	-8.2180
66	Comb4	155.0430	-0.0430	-7.4840

CHAPTER 6

CONCLUSIONS AND FURTHER WORK

6.1 Conclusions

This study utilized for optimization performance of frame structures by interfacing GA optimization and SAP2000 FE analysis programs.

The optimization method used is useful creative design aids for structural engineers. It allows a reduction in weight and the optimal structures obtained by introducing the cross sections as well as their properties according to more actual constraints based on some general design codes.

Design methodology of space moment frame that combines stiffening sizing optimization has been an important role in minimizing the amount of material used in the construction of the structure for economic point of view.

Optimization algorithm starts by the implementation of the analysis of the structure. A GA-SAP2000 program which uses the FEMs based numerical analysis was combined. To achieve size optimization based on genetic algorithm to perform the analysis and design of the space frame which is a complex structure in a 3D system.

The introduction of moment frames as well as section variation leads to a significant improvement in the objective function as demonstrated by several examples.

Several models are analyzed and optimized based on stress and total story drift constraints successfully, and the results are compared with the other researches.

The use of nonlinear finite element SAP2000 commercial program can assist greatly in achieving a safe design and is used to check if the applied inner force, member groups and elected sections are corresponded specified code and constraints or not.

However most of these commercial packages have been developed to be used as verification rather than the optimization tool, but it is possible to do it by designing an optimization code in an open file to achieve this task and interfaced with them.

6.2 Future Work

Continued research is allowed for combining with other FEM programs and optimization methods.

Use the GA-SAP2000 optimization technique in the form of parallel computing by using more than one computer instead of one to rapid the convergence time.

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