Optimum Design of Transmission Towers

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ABSTRACT

OPTIMUM DESIGN OF TRANSMISSION
TOWERS

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This thesis deals with the improvement of reliable and efficient computational tools to analyze and find optimum weights for transmission towers subjected to static loading. The finite element method is used to determine the stresses and displacements. An automated analysis and optimization procedure which integrate with finite element analysis and numerical methods have been used. Also, a FORTRAN based genetic algorithm is implemented to search the optimum design. Finally, the results that are obtained from these programs are compared with the available literature and SAP2000 to figure out the efficiency of FORTRAN program.

Key words: Transmission tower, static analysis, optimum weight, finite element method.

ÖZET

Enerjı Nakil Kulelerinin Optimum
Tasarımı

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Bu tez, statik yüklemeye maruz enerji nakil hattı kulelerinin analizi ve optimum ağırlıklarının bulunmasına yönelik güvenilir ve verimli bir bilgisayar programı geliştirilmesini kapsamaktadır. Sonlu elemanlar yöntemi kullanılarak gerilme ve şekil değiştirme değerleri hesaplanmıştır. Sonlu elemanlar yöntemi ve sayısal yöntemleri birleştiren bir otomatik analiz ve optimizasyon tekniği uygulanmaktadır. Optimum tasarım için FORTRAN tabanlı bir genetik algoritma kullanılmıştır. Son olarak, FORTRAN programının verimliliğini ölçmek amacıyla elde edilen sonuçlar SAP2000 ve önceki yayınlarla karşılaştırılmıştır.

Anahtar kelimeler: Enerji nakil hattı kuleleri, statik analiz, optimum ağırlık, sonlu elemanlar metodu.

To My Family

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LIST OF SYMBOLS

Abbreviations:

2D	**T**wo **D**imensional
3D	**T**hree **D**imensional
ASD	**A**llowable **S**trength **D**esign
AISC	**A**merican **I**nstitute for **S**teel **C**ostruction
DOF	**D**egree **O**f **F**reedom
DOFs	**D**egree **O**f **F**reedom**s**
DL	**D**ead **L**oad
EL	**E**arthquike **L**oad
FE	**F**inite **E**lement
FEA	**F**inite **E**lement **A**nalysis
FEM	**F**inite **E**lement **M**ethod
GAs	**G**enetic **A**lgorithms
LL	**L**ive **L**oad
MSM	**M**atrix **S**tiffness **M**ethod
MSA	**M**atrix **S**tructural **A**nalysis
NJ	**N**umber of **J**oints
NR	**N**umber of Joints **R**estrained by Support
SFEM	**S**tochastic **F**inite **E**lement **M**ethod
SO	**S**tructural **O**ptimization
WL	**W**ind **L**oad

Latin Symbols:

Scalar

A	area
$C(1)$	order of continuity
$c_{i,j}$	normalazied value of i^{th} constraints for j^{th} population
C_c	Corresponding Critical value
C_b	bending coefficient
d	displacement
d_{all}	allowable displacement
E	Young's modulus
$F(\mathbf{x})$	objective function to be minimized
$\hat{F}_j$	penalized objective function
$F_{j(best)}$	fittest design variable
$\bar{F}$	average fitness value
F_a	allowable axial stress
F_b	allowable bending stress
F_v	allowable shear stress
F_y	yield stress of material
F_{ob}	elastic lateral torsional buckling
f_v	shear stress
G	modulus of rigidity
$g_{i,j}$	i^{th} constraint function of j^{th} of population
$\bar{g}_{i,j}$	limit of normalized contraints
$\tilde{g}_{i,j}$	Penalized constraints being violated
$\hat{g}_j$	sum of penalized contraints
$g_{i,all}$	allowable value of i^{th} constraints
$g_j(\mathbf{x})$	inequality constraint function
k	the degree of constraints violation
l_{22}, l_{33}	major and minor direction unbraced member lengths
lcrom	chromosomes length
I	moment of inertia
J	polar moment of inertia

k	nodal displacements differentiating twice (curvature)
K	effective length Factor
L	length of element
M_a, M_b	end moments of unbraced member
N_i	shape function associated with node *i*
N_{gen}	number of generation
n_c	number of constraints
n	sampling points for gause legendre
P_s	population size
P_c	penalty coefficient
Q	reduction factor for slender section
r	resolution (intermediate value) in continuous optimization
r_{33}, r_{22}	radius of gyration in major and minor directions
r_z	minimum radius of gyration
r_o	polar radius of gyration
S	section modulus
u_x, u_y, u_z	displacement components in x, *y* and *z*-direction
U	internal energy
X_{max}	maximum limit for design variable
X_{min}	minimum limit for design variable
W	external energy
W_i	Weight for Gause Legedere
x_i, y_i, z_i	typical cartesian coordinates of node *i*
x, *y*, *z*	global cartesian coordinates
x_o, y_o	coordinate of the shear centre

Vector

$\mathbf{R}_{xi}$, $\mathbf{R}_{yi}$	force components in the local x, y direction for element of node i
M_i^e	moment at the i node of the element (e)
$\boldsymbol{\delta}_{xi}$, $\boldsymbol{\delta}_{yi}$	displacement components in the local x, y direction for element of node i

θ_i^e	rotation at the i node of the element

Matrix

[B]	strain-displacement matrix
[D]	matrix of rigidities
[J]	Jacobian matrix
$[\boldsymbol{k}'_{\mathbf{a}}]$	local stiffness matrix for axial load element
$[\boldsymbol{k}'_{\mathbf{b}}]$	local stiffness matrix for bending case element
$[\boldsymbol{k}_{\mathbf{g}}]$	geometric stiffness matrix
$[\boldsymbol{k}'_{\mathbf{T}}]$	local stiffness matrix for torsion element
[N]	shape function matrix
[r]	one node end force transformation matrix
[T]	transformation matrix

Greek Symbols:

α	angle between local and global axes
β_w	special section property for angle
Θ_x, Θ_y, Θ_z	rotation components in local *x*, *y* and *z*-direction
ν	Poisson's ratio
ξ	isoparametric element natural (curvilinear) coordinates
σ	stress component
σ_{all}	allowable stress
ρ	mass density
γ_x,γ_y,γ_z	global components of material weight
Π	total potential energy

CHAPTER 1

INTRODUCTION

1.1 General

Latticed structures are used in a wide variety of civil engineering applications. A latticed structure is a system of members (elements) and connections (nodes) which act together as a one structure to bear and resist the applied acting load. Transmission line as lattice structures plays a significant role in human society and a failure in the system of transmission line may cause an interruption in the energy supply to consumers, which may lead to severe social and economic losses. To prevent this occurred it must guarantee the stability and the low cost of maintenance, and this system must be well designed.

The structure cost usually accounts for 30 to 40 % of the total cost of a transmission line. Therefore, selecting an optimum structure becomes a crucial part of a cost-effective transmission line design. A structural study usually is performed to determine the most suitable structure configuration and material based on cost, construction, and maintenance considerations and amount of electricity [1].

The used structures on transmission lines are generally lattice or pole type. Lattice type structures are commonly composed of angled steel sections. Pole types may be concrete, wood or steel. Both types of structures are self-supporting or guyed [1].

The minimum weight of the tower structure maintained by using the minimum cross sectional area for the members under the constrained condition to make members full stressed according to the allowable tolerance of the material used for fabrication of the sections.

Nowadays computers are playing an important task in the analysis and design of such structures. In the past analysis and design were performed by manual calculations based on a two dimensional (2D) stress analysis which is time consuming and laborious. The highly sophisticated softwares have been developed to automate calculations of member forces based on three dimensional (3D) Finite Element (FE) analyses. Such softwares find out critical member forces for a type of loading and a variety of possible tower combination, giving accurate results for analysis and design.

Optimization is an automated design procedure in which the computers are utilized to obtain the best results. The numerical methods of Structural Optimization (SO), with applications of computers automatically generate a near optimal design (converge to solve) in interactive manner. A program was modified and used to automate analysis and optimization of the structure written in FORTRAN language based FE analysis and Genetic Algorithm (GA) optimization technique.

1.2 Transmission Towers

Transmission towers were increasingly constructed for transport high voltage from the source to consumers. Because of it's advantageous to reduce the number of locations from a maintenance point of view (longer span) to get proper ground clearance and to have strong support to withstand wind load. The tower is a balanced structure with four legs covering the spans of 250 meters and above can be adopted for tower line.

1.2.1 Parts of transmission towers

As shown in Figure 1.1 transmission towers are consist of a pylon with crossarms connected by hanging insulator for supporting conductors and at the top earth wire with extra accessories like spacer and vibration dampers.

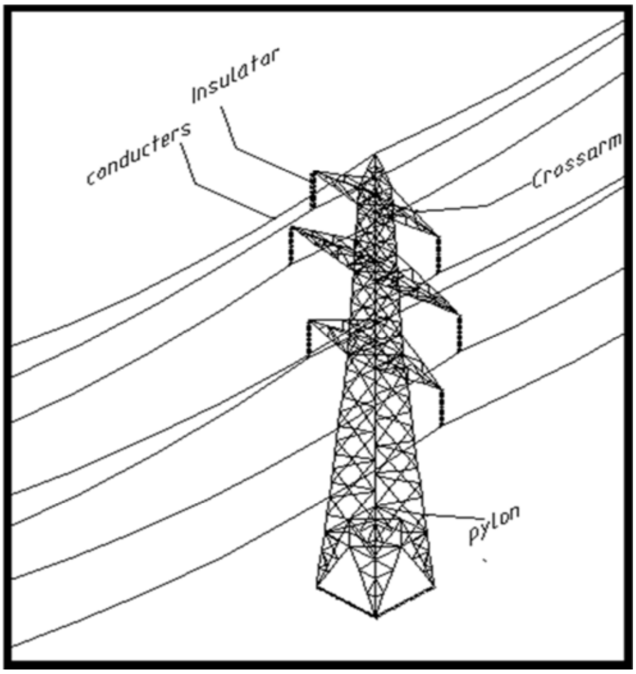


Figure 1.1 Transmission line tower.

1.2.2 Transmission towers types

There are many classifications for types of towers, which depends on,

- Number of used circuits,
- Use in the line alignment (such as; straight, varying angle and dead ends).

Generally transmission towers in line alignment classified to many standard types as mentioned below:

Tangent suspension towers: straight line and deviation angle up to about 2°. These towers are designed to withstand to the, ice load, wind load and broken conductor loads. Hence 90 percent of the lines are of this type and the SO tools become important to reduce the total weight of the structure under service condition.

Angle towers (semi-anchor towers): deviation angle greater than 2°. They must withstand the transverse load from the line tension and its components produced by this angle. Mainly for angle towers are classified into the types below [2];

- Light angle: from 2° to 15° angle of the line deviation.

- Medium angle: from 15° to 30° angle of the line deviation.
- Heavy angle and dead end: from 30° to 60° angle of the line deviation.

1.2.3 Transmission tower materials

• **Metal:** galvanized steel and aluminum rods, bars and rolled shapes, fabricated plate and tubes.

• **Concrete:** spun with pre-tensioned or post-tensioned reinforcing cable, statically cast, non tensioned reinforcing steel and single or multiple piece.

• Other types of wood as grown and glued laminated, plastics, composites and variation of all the above.

Depending on their style and material contents, structures vary considerably in how they respond to loads. Some are rigid or flexible. Those structures that can safely deflect under loads and absorb energy while doing so, provide an ameliorating influence on progressive damage after the failure of the first element [2].

1.2.4 Factors affecting structure type selection

There are many factors that effect on the selection of the structure type for use in a transmission line. Some of these which have more influence are shortly identified below:

Erection technique: It is clear there is many require different erection techniques for different structure types.

Public concerns: Perhaps the most difficult factors to deal with arise as a result of the concerns of the general public living, working, or coming in proximity to the line.

Inspection, assessment and maintenance: Taking this point in view for future maintenance when there is a cut or the type of the equipments for easy reach and working on the alignment of the transmission line.

Future upgrading or updating: Because of the difficulty of procuring the right of way's and to get the necessary permits to build new lines, many facilities make improvement for their future options by selecting structure types of current line projects that will permit future upgrading or operating initiatives [2].

1.3 Design Span Lengths

When transmission towers are designed, there are many terms used for span in calculations according to the location of the tower on the line alignment such as:

- Normal span: the line is designed over level ground, so that the required ground clearance is obtained at the maximum specified temperature. The normal span is the most economic span.

- Ruling span: ıt is a weighted average span of the varying span lengths the assumed design span measured between dead ends and it can be calculated by the equation:

$$\text{Ruling span}=\sqrt{\frac{L_1^3+L_2^3+\ldots L_n^3}{L_1+L_1+\ldots L_n}} \qquad (1.1)$$

  Where L_1, L_2, $\ldots L_n$ are the spans Lenghts consequently in longitudinal section of line alignment. The horizontal component of tension is found with the application of this span in calculations and to be used to all spans between the anchor points.

- Average span: is the mean span length between dead ends. On this assumption the calculation of sag and tension is taken, where it is expected that freely suspended for the conductor in a way for every separate span gives varies replies in tension as a single average span.

- Wind span: is that span which is calculated from the average of the two spans adjacent to the support is assumed to work transversely on the conductors.

- Weight span: can be measured as the horizontaly distance between the lowest sag points of conductor for the two adjacent spans from the tower. This span is used in the design of the arms [3].

1.4 Basic Idea and Theories Used in Analysis

The basic theories used in the analysis are the following: Matrix Stiffness Method (MSM), the possible displacements of the ends Degrees Of Freedom (DOFs), the global and local coordinate systems.

1.4.1 Matrix stiffness method

The recent effective technique in the analysis of engineering structures is MSM. The presence of effective computer programs and modern computers supplemented efficiency and common function of its applications. By taking into account different features of a structure and loading MSM permits carrying out detailed analysis of every complex 2D and 3D structures in engineering. A set of new ideas is required for the method. They are possible displacements of the ends, global and local coordinate systems, FE, separate element stiffness matrix and whole structure, etc.

The scheme of the displacement method is used by this method and covers its extra developments. The presentation of arbitrary structure should be taken as a set of FEs and three aspects of any problem such as, statically, geometrically and physically should be shown in matrix form. The bending moment diagram caused by unit primary unknowns in the primary system does not require in MSM method. Instead it is needed to prepare few initial matrices according to strong algorithms and perform matrix procedures by computer using the standard programs [4].

1.4.2 Degree of freedoms and displacement of joints

When a set of FEs representing a structure, the description of the possible end displacements of each member is needed. These displacements are representing the unknowns in the displacement method. These displacements are known as the possible angular and direct displacements of the joints. The term of possible involves that in a particular structure such displacement is possible, but not fundamentally used. For example, in case of two span continuous beam, a section at the middle support generally rotates; but, if symmetrical beam and loading are available then the rotation angle is zero. In brief, we can consider it that any possible displacement of the joint is a displacement in global coordinates.

1.4.3 Coordinate in local and global systems

The local coordinate system is indicated to as the specified element, while global system is associated to the whole structure. The relation between the global and local coordinates is done by apply the transformation matrix with trigonometric function.

1.5 Analysis and Design of Steel Frame Structures Related Problems

The strategy for structural steel construction is a procedure based on many contributing sides. Past practice of successful and unsuccessful structure, laboratory experiments and outcomes of research, come together to confirm structures do not collapse. Structures can then be used powerfully and securely but on the other hand must be economically built and maintained. Since it can be known that the design process must satisfy two opposing goals economy and security. Performing this compromise is not an easy job, therefore codes of practice have developed to help and guide the designer, but special national codes, for example, British and American codes of practice deal with the design problem differently. This may be because the behavior of steel construction frames, for example, is not good understood because methods of design are still at an elementary stage of improvement. This may be due to the reality that the problem of design is much less specific than that of analysis [5,6].

The problem of design or composition comprises producing member sizes which are acceptable in all requirements, under all loading conditions. In most cases, an unrestricted number of designs will meet these requirements. More practical designs are based harshly on a trial and error process. The design process starts with the analysis step. After analyzing a structure, an initial solution of the member properties is modified and the next solution is re-analyzed.

1.6 The Requirement of Design Optimization

When conventional design methods are used design engineer may face some severe difficulties. Firstly, the talent and practice of the designer is important to obtain safe designs. Secondly, the complication of the treated structure and doing a number of re-analyses and following redesigns are very tedious and probable cause of mistakes. Thirdly, there is the trouble of covering all probable loading cases. Fourthly, the proposed usage of the structure might avoid the designer from getting economical design. Fifthly, the different design and analysis techniques might hesitate the designer in choosing the appropriate technique. Consequently, the use of computers offers reliable and precise analysis much easier. Also the speed of computer application is faster compared to traditional hand solutions.

Design optimization is a stimulating research topic and proposals for design optimization have been made by design professionals. Design optimization is focused with the problem of the selection of geometric parameters and mechanical strength properties of the structural elements. This choice involves of a search for the extremely solutions, which fulfils the specified criteria, the investigate being shown in an objective and reasonable way, that does not depend on the feeling or special skills of the designer. Thus, design optimization takes over that part of the design process, which consists of choosing sizes and later checking that the demanded criteria have been met. The question arises whether the design optimization field can or should fully substitute conventional designing ways, for example, whether or not the task of optimization is to adopt all structural parameters so that the solution of an optimization problem should be the same to attaining a complete design of a structure [5,6].

1.7 Layout of The Thesis

In present work main attention is focused on SO of lattice transmission towers under static loading condition. To do so FORTRAN based analysis and optimization tool is improved for using in this study. The main goal of the study is to reduce the total weight of transmission towers without causing a strength base failure. The organization of the study and the layout of the thesis is now pronounced:

- Chapter 2 is the literature review in analysis and design optimization.

- Chapter 3 is devoted to the static analysis of 3D structures. The basic theory and matrix analysis is first presented and then several examples are studied and illustrated.

- Chapter 4 deals with various aspects of the optimization process including the definition and selection of the design variables and the GA technique.

- Chapter 5 presents the work of SAP2000 software in the design of space truss with the (AISC-ASD-89) code. The optimum structure is designed under the auto selected section feature in the software.

- Finally in Chapter 6, some brief conclusions are presented together with some suggestions for future work.

CHAPTER 2

LITRATURE REVIEW

2.1 Literature Survey

The analysis and optimization of lattice structures have found great attention in literature. Different analysis methods were used by researchers to capture the response of lattice structures under service loads. The analysis techniques which are used for lattice structures were also used for transmission towers. So it is important to capture the works presented for analysis of transmission towers to link theoretical knowledge and the experiences on lattice structures.

Also it is important to detail published works on the SO to compare performances of approaches. This chapter treats the works published so far on structural analysis and optimization of the lattice structure.

2.1.1 Analysis

During the past decades, before the advent of computers huge experimental and theoretical investigations on such structures have been carried out and the evolution of Matrix Structural Analysis (MSA). The researchers performed on MSA from 1930 to 1970 is outlined [7]. With the available tools it was very difficult to analyse 3D trusses. Therefore, simplifying assumptions were made in order to reduce the analysis of a tower to independent analysis of several statically determinate plane trusses. These assumptions in structural analysis are explained by Marjerrison in

1968 and Zar and Arena in 1979 [8] to simplify the analysis as determinate plane truss using algebraic or graphs.

Of the many existing analytical and numerical tools, the Finite Element Method (FEM) has been the generally common method used in the analysis of transmission tower. Generally, the stiffness matrix method is employed in the model of the transmission tower. Although, with the rapid progress in high-speed computation, because of the huge amount of degrees of freedom required for exact modelling, described 3D FE. Analysis of transmission tower is still time consuming because it needs large data input and it produces enormous output with solving very big dimension matrices, they frequently show the natural behavior of the transmission tower.

The principal of the stiffness matrix was illustrated early between 1954-1955 by Argyris [9]. These articles were originally published in a series of articles in aircraft engineering. The purpose of these papers is to generalize and extend the fundamental energy principles of analysis of elastic structures. The most important contributions of his study are the matrix methods of analysis.

Jensen et al. [10] presented a paper about analysis of self supporting structures as highly indeterminate structures. Such structures were solved by determinate methods due to the number of calculations involved in indeterminate methods. Large numerical problems were solved with digital computers, for this the researcher presented a technique for efficiently solving large systems without resorting to matrix inversion.

Lee et al. [11] presented a method for the limit analysis of indeterminate space truss utilizing the static equilibrium equations and linear programming techniques. In that paper, a general formulation for both the limit analysis and design problems were developed. They mentioned that additional research is required to enhance the future use of limit design for the transmission structures.

An incrementally small-deformation theory that was physically self explainable is presented by Yang et al. [12] for the large displacement nonlinear analysis of structural frames. They made them assumptions strictly based on small displacements, small rotations, and small strains during every incremental stage, from the force displacement relationships the elastic and geometric stiffness matrices for the beam element are derived.

The structure's stability is associated with the actual behaviour of being have a probabilistic description of close to semi-rigid connections instead of the supposed hinged connections. For this objective Silva et al. [13] presented a paper that suggests a different structural analysis modelling approach to the design of steel tower take into account completely the real structural moments and forces linking 3D truss and beam FEs. The two mentioned design methods was compared with a third method established for the use of spatial beam FEs to model the main structure and the bracing system on two actually built steel telecommunication towers (40 and 75 m high steel towers) are explained.

The objective of the paper prepared by Lee et al. [14] was to expand a numerical model for representing the ultimate behavior of lattice steel tower structures. The elastoplastic big deformation analysis of a lattice steel tower structure by FE analysis were displayed and the numerical results were compared with full scale destructive tests. A 2-node 3D L-section beam FE is used and the eccentricities of loading and boundary conditions were considered as well as material and geometrical nonlinearities. The real tower structure model was used and built with sections using the beam elements.

The objectives of giving a presentation of past and recent developments in the stochastic FEM area and signifying future directions as well as some open issues to be checked by the computational mechanics community in the future. Consequently Stefano [15] mentioned in his article that an extension of the classical deterministic FE approach to the stochastic framework was SFEM (i.e. with stochastic mechanical, geometric and/or loading properties in the solution of static and dynamic problems). They were addressed some open issues and indicating future directions to be studied by the engineering community in the future.

Taking into account the real lattice towers complexity, also it would be important to achieve investigations on a real tower in the field to give owners and designers on the stiffness and damping of lattice towers with full information. A study was presented by Taillon et al. [16] dealing with the design of steel lattice structure, although the dynamic loads are controlled this structure such as shocks (free vibrating) resulting from broken components such as guy cables or conductors, earthquake, or wind. In the study, found the practical information on modeling techniques to be used for lattice structures. They show that the stiffness and damping relation to evaluate the influence of load level on both structural characteristics. The 40% of stiffness can be reduced and the values between 2 and 5 % of damping ratio can reach. The structural models of three types have been used to the tower to predict the initial stiffness. When the boundary conditions taking into account it was a very important assumption in proper modeling of the tower. In terms of modeling technique they recommend, also it would be interesting to examine the overall behavior and the interest of nonlinear analysis with the effect of the eccentricity of connections.

Transmission tower structures with nonlinear analysis technique has been presented in a study by Albermani et al. [17] They proposed a procedure to be used with exactly expect structural failure, Their expectations in the study are based on of an expensive full scale test results. This accuracy is given, and the method that applied to failure analysis and expectation, and for design upgrades and modifications. An enormous savings in supplies by using of this method is obtained, and will reduce the need for the full scale testing that is ordinary in the transmission industry. Consequently, modified and upgraded can be easily done for tower designs.

The analysis of space frames with the FE formulation is discussed by Shakourzadeh et al. [18]. The joint connections deformation in linear, non-linear and stability analyses of 3D thin walled beam structures to be considerd a numerical method was presented. The consideration of joint deformation caused by a torsion, membrane, bending or even warping effects are taken. An existing FE code can be easily introduced in their method. Determining the overall stability and ultimate strength response of framed structures has been demonstrated with the importance of connection behaviour. The response of frames is very sensitive to the joint flexibility as the numerical examples presented.

The joint effects were described for many models representation and the numerical predictions were compared with experimental results as objective discussed by Jiang et al. [19]. The importance in design decision making is an accurate expectation of the ultimate load capacity of transmission towers. The conventional structural analysis models which do not take into account joint eccentricities and slippage effects are non-conservative in predicting the global response of lattice towers measured in full scale tests. They reach in their study to the following decisions that in numerical models both diagonal members and main leg splice connections by the effects the joint slippage can expect the tower displacement with reasonable engineering accuracy. When under the consideration of the joint slippage increases the expected tower distortion without changing its failure mode. In the other hand the effect of joint slippage on the ultimate load bearing capacity of the towers will be find by the magnitude of the applied vertical load and by the load directions and associated tower failure mode.

2.1.2 Structural design optimization

In this section the literature is observed and from the study is classified with related studies work with GAs or traditional method.

2.1.2.1 Structural optimization using genetic algorithms

GA which has been used for many years is one of the SO methods. This method is also used as a tool to optimize lattice towers in this study. GAs, firstly examined by Holland at the University of Michigan as search procedures based on natural selection is the basic engine of Darwinian natural selection and survival of the fittest [20]. The literature is reviewed for the GA, and this literature classified into three types of studies:

2.1.2.1.1 Specialized design

Specialized structure designs involve GAs that adapt to particular structure types. Some of these designs will mainly include trusses, 2D frames, 3D frames and lattice towers.

The paper presented by Ghasemi et al. [20] demonstrated the use of GAs for SO of trusses. They discussed the idea of rebirthing to be significantly efficient for problems concerning continuous design variables. Specific standard examples were examined concerning 4-bar, 10-bar, 64-bar, 200-bar and 940-bar 2D trusses. Together continuous and discrete variables were studied. A normal GA technique and other improved one were presented and their implementation has been examined with some 2D plane trusses. Together approaches provide fine solutions when compared with results existing in the literature with the modified GA technique providing an improved total implementation. The application of rebirthing lets improve optimal solutions to be achieved for problems concerning continuous design variables supplied that the solution at the first rebirthing is near to the optimal solution.

A design procedure incorporating a simple GA is developed for discrete optimization of 2D structure is included in article by Camp et al. [21]. The objective function of the research was the weight (or cost) which is minimized in correlation to the serviceability and strength requirements. To complete this optimization, a program was developed based on a simple GA for optimization called FEAPGEN. This program was a module incorporated into a modified version of a FE analysis program. This program includes special features such as discrete design variables, open format for designed constraints, design checks using AISC-ASD specification, and multiple loading conditions.

A paper was reported by Erbatur et al. [22] which was discussing the expansion of a discrete optimal design of planar and space structures by computer-based systems approach for composed of one-dimensional elements. The important characteristic of the solution approach was the application of GA in the optimization process. The steel frame and truss structures applications and experience were discussed. The comparative studies results of the GA against various discrete and continuous optimization algorithms for a group of representative structural design problems were described to display the efficiency of the former. It was performed that GA regularly finds the region of the search space including the global optimum, but not the true optimum itself. Moreover, in this study a method based on a suggested multilevel optimization was tested and showed to overcome this fault.

The minimum weight weight optimization as optimization process was presented for the optimum weight by Tong et al. [23] discussed the truss structure subjected to constraints on stresses, natural frequency responses using the discrete design variable. The study consisted of two steps for optimization procedure. The first step is to use a difference quotient method and find a feasible basic point through defining a global normalized constraint function. The second step is to establish the discrete value of the design optimization into linear zero-one programing. They employed a binary combinatorial algorithm to achieve the zero-one programing. Examples of discrete optimum truss design were presented to demonstrate the feasibility of the optimization process.

The paper by Domingues et al. [24] investigated the topology optimization with geometry and simultaneous size of existent great truss structures using GAs as optimization technique and the analyser as the FEM. Thus, when optimizing the characteristic of basic bays instead of whole structure to get the final optimum design may be reached. The cross-sectional areas have been obtained from the standard profiles according to AISC codes in order to obtain realistic optimal designs, and practical conditions are imposed on the bays. Also the constrained by the maximum and minimum cross-sectional area, maximum slenderness ratio and the maximum stress of the truss members in the design optimization problem. To take all these constraints in account, the consideration of two different penalty functions was taken. The first penalty function with respect to the normalization of violated constraints considering the allowable stress or slenderness ratio. The second penalty function is a constant function which is worked to penalize the violations of the slenderness ratio. Illustration of the method was done on the 2D and 3D examples to show the performance of the study.

2.1.2.1.2 Genetic algorithm improvements

Improvements include the research done to enhance the robustness of the optimization program. This classification will include research improving execution time, crossover techniques and selection.

The observations were made on the discrete optimal design because they note that almost all design variables in many of SO problems were discrete in nature using the proposed GAs. This feature makes it a perfect choice for optimization problem, when the optimum solution produced will be feasible from both a calculation and practical point of view. Rajeev et al. [25] found that the number of function evaluations is greater, hence the number of analyses. Though the gradient computations were absent, GAs is slower compared to traditional optimization algorithms. This problem is not viewed today as a limitation in the computing environment, when fast computers are available with relatively low cost.

Classical optimization method using GAs during recent years as a valid option, such as heuristic algorithm, especially using with the problems have huge cardinal searching space, very common incomplete problems. The goal of the work which was presented by Barrios et al. [26], the study of the necessary conditions (measured as the optimum values for the GA main parameters) to obtain the convergence in the shortest possible time. In order to reach the aim of the study, a dynamic model, based on biological models of evolution was proposed. The differential equation could be defined The model, that they were study to determinate the conditions that enable the study to ensure the convergence of the algorithms and the conditions for accelerating this convergence.

Chen et al. [27] focused on the search for the most economical steel roof truss design in a reasonable amount of time with the improvements of a design software system that has enough flexibility and capability. Improving the efficiency and robustness of the GA approach that developed earlier was the objective to be achieved. The influences of schema survival, schema representation, problem definition, type of crossover, the number of design iterations, and size of the population on the computational expense and the value of the objective function are considered. Their results show some interesting conclusions, that the maximum number of design iterations (generation) and the size of the population need to be at minimum the size of the chromosome. The one of the most important factors was the schema representation. The initial design (density of the ground structure) and the size of the chromosome was depending on the complexity, a newly developed association string strategy has led to a computationally effective GA process when combined with the elitist, one-point and uniform crossover strategies.

The crossover was one of the basic three operators in any GA. A study was proposed by Hassançebi et al. [28] with several crossover techniques and their relative specifications under investigation. And they performed that the mixed crossover use displayed a quite suitable performance for all test problems and they observed from the test examples that the success of the GA method increases with the size of the design space is increased, hence the design space is necessary to search the optima for the solutions which is the best solution taken by the mixed crossover used method.

The adaptive approach in GA was discussed in articale by Toğan et al. [29]. In this study they attempted to show how the implementation of GAs was influenced the adaptive method, and some improvements were proposed in each of crossover, mutation, and penalty function. An approach was taken into account for grouping of members to reduce the size of the problem. Space truss with several practical design examples taken from the literature were optimized by the algorithm suggested in the work. Design constraints such as stability, displacements and tensile stress given by national specifications were included and the comparison of results with that ones achieved by previous studies. In the study the probability of catching the global solution and enhance the performance of GAs it was increase by concluded the member grouping together with the adaptive approach.

Vedat et al. [30] showed that many factors such as genetic operator, coefficients, and some strategies effected on the performance of GAs. These strategies also affected by the initial population and member grouping as samples of factors. When the initial population is applied to minimize the number of searches to reach the optimum design variable in the search space, hence grouping strategy is performed to reduce the size of the problem. They proposed that this strategy reduces the number of searches within the solution space and effectively performed the convergence capability and directly the performance of the GA.

The design variables encoding type such as value and binary encoding is important to save more memory for computation this study concerned by Dede et al. [31].

2.1.2.1.3 Optimization goals

Optimization goals focus on the GAs designed specifically to improve the optimization of one or more objectives. Some of these objectives include member size, shape, topology, and vibration control.

The various developments of increasing complexity involved in layout optimization were discussed by Azid et al. [32]. They mentioned the application of traditional GA in layout optimization briefly with draw attention to its conditions and limitations imposed to finding the optimal design. They suggested a new method applied to the benchmark examples for verification. The methodology also was used to new examples of crane truss and bridge truss problems so as to determine the generality and robustness for topology optimization. They extended their method to involve dual stress displacement constraints then many practical problems include these two constraints at once. Also they focus on the effect of mutation on the final topology. The explanation of the robustness, generality, and ability in obtaining optimal designs by using the suggested new method in layout optimization problems.

Shape optimization through a GA using discrete boundary steps and the fixed-grid FE analysis concept was recently introduced by the Woon et al. [33].

The paper prepared by Göğüş et al. [34] deals with the SO of vibrating planar and space trusses. The objective of the study was to develop a robust and reliable shape optimization tool for vibrating trusses. Natural frequencies are determined using standard FE matrix displacement method. The fundamental concept is to generate structural shapes for trusses in which certain vibration characteristics were improved.

The comparative studied results of the GA against other continuous optimization algorithms for truss examples were described to display the efficiency of the SO.

The transmission tower shape can be optimized by a GA to acquire the maximum fundamental frequency, considering that there are some disadvantages such as premature convergence and low robustness in solving complex optimization problems with the standard GA. In connection a new adaptive strategy was suggested by Gan et al. [35] to improve the performance of the algorithm, including the design for selection mechanism and the method of selecting both dynamic crossover and mutation probability. Then they show solution for several problems in independent optimization program. Finally, they showed the efficiency and feasibility of the method with solving a tower with 108 bars.

2.1.2.2 Structural design optimization traditional methods

In this section the optimum design of structures (2D and 3D) by traditional methods in literature is presented. This section is in relation with practical methods submitted by the researchers to compose the designer's with economic design and rich the literature with this type of the study.
Rao [36] discussed the concept of optimum design beside transmission towers with practical design controls. In the study for obtaining minimum weight of tower design in both crisp and fuzzy environments a systematic procedure has been presented. The chosen of a few parameters to enhance the optimization scheme that influences the tower weight indirectly and geometry, by means of design control variables. On the other hand it reduces the computational effort significantly. They devoted a program has been improved for developing the optimum design. The study covers the result of the typical transmission tower, and presents substantial saving in tower weight.

The discrete variables linked to the problem of optimal structural design was presented by Huang et al. [37]. When selecting a discrete 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. In their study they made three approaches that link a 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. The problems solution with discrete variables taking into account the CPU times are large. They proposed an approach to reduce the CPU times.

The examining for the best of all combinations is the basis of most optimum structural designs, arising from the parameters of listed rolled profiles, and the number of structural members. Consequently Blachowski et al. [38] adopted method on this idea, because completely known methods of finding the discrete minimum of structural weight require a very large number of analyses often of an order of four. In their study, for solving such problems a relatively simple method was presented. A tree graph was the basis of it, demonstrating discrete values of the structural volume. The structure could be exposed toloadings of multi static with the constraints imposed on stresses and displacements. The knowledge of apply the method is limited to graph representation and the FEM. They explained with two problems has a numbers of combinations up to 42^{38}.

2.2 Summary

The literature is reviewed with available articles related with the topic of the study. When the information about the study is collected, after that these topics are classified related to the nature of each study. Firstly we listed the works which deal with analysis in the first section to show the study adoption analysis of the truss structure generally and connections particularly. Secondly, the next section is related to the design optimization of truss structure. These classified articles are linked to zero-one algorithm according to classification have taken from the nature of the study. Finaly the section cover the literature of traditional methods for design optimization.

CHAPTER 3

STATIC ANALYSIS

3.1 Three Dimensional Space Frame

Space frame is the common to the most types of the framed structures which have members oriented in any direction in the 3D space. 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.

Where the analysis of space frames is generally based on the assumption that the cross section areas of the all members are symmetric about the two mutually axes and frees to warp out of their planes under the action of torsional moments.

The DOF of a structure are the independent joint displacements (translations and rotations) that are necessary to describe the deformed shape of the structure at the time when subject to random loads.

$$NDOF=6\ (NJ)\ -NR \qquad (3.1)$$

Where NDOF is the number degree of freedom for 3D elements, NJ is the number of joints and NR is the number of joints restrained by the support of the structure [39].

3.2 Theory of Structural Analysis and Stiffness Matrix Formulation

Essentially the direct stiffness method is the basis of all modern structural analysis softwares, which is a branch of the FEM. Now, a group of elements and nodes is measured as the structure. A location in space is expected for every node, outlined by a set of degrees of freedom and the coordinates. DOFs are the elements of the displacement vector (selected as u_x, u_y, and u_z in 3D space) and the components of a rotation vector (selected as θ_x, θ_y and θ_z). With each DOF is a relating moment or force that, after multiplied by the DOF, gives a units of work [40].

3.2.1 Stifness matrix method

Stiffness matrix formulation is done by the principals of virtual work which is the area under the force-deflection curve and according linear behaviour, displacements and forces are proportional by deflection, where single forces varies linearly with displacement from zero to its final intensity F1 as shown in the Figure 3.1 [41].

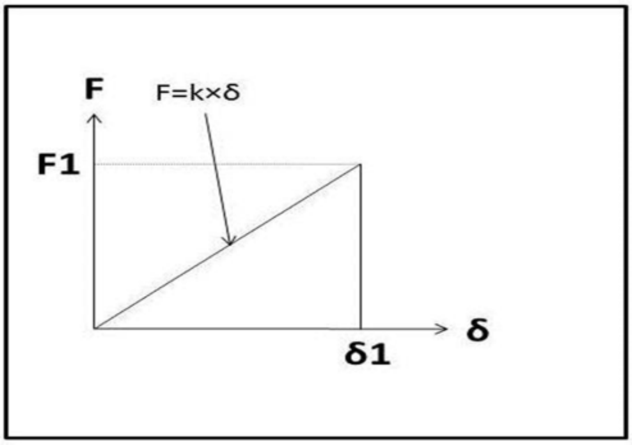


Figure 3.1 Force deflection relation.

Where from the Figure 3.1 by calculating the area which is triangle to represent the work done by F1 can be written as :

$$W = 1/2 \times F1 \times \delta 1 \qquad (3.2)$$

3.2.1.1 Two dimensional frame element

The nodes are the connection of elements, and every element supplies stiffness with relate applied forces (or moments) occurring related displacement (or rotation) DOFs of the nodes it connects. Since multiple DOFs exists for a structure element, there are many stiffness coefficients. Setting the stiffness coefficients into a stiffness matrix with the relation vector of each DOFs for the nodes of the element to a vector of each relating force and/or moment components for the nodes of the element. Stiffness coefficients are generally set in terms of material properties and cross section of the element. The general form of the equation provided is $f^{(e)} = k^{(e)} u$, which is an equation of equilibrium that shows "external applied forces, f are balanced by internal forces, ku". Note that stiffness matrices for structural elements are normally defined in a local frame of reference, defined to pass along the length of the element.

The formulation from 2D element is extended to 3D element in the space when the joints are rigidly connected a typical member for 2D element is shown in the Figure 3.2 where the displacement and forces take action on the element ends as shown we have three independent displacements for the results of acting forces at each end which can be written below [39],

$$R_i^e = \begin{bmatrix} R_{xi} \\ R_{yi} \\ M_i \end{bmatrix} \quad , \quad \delta_i^e = \begin{bmatrix} \delta_{xi} \\ \delta_{yi} \\ \theta_i \end{bmatrix} \qquad (3.3)$$

Where R_{xi} , R_{yi} and δ_{xi} , δ_{yi} are representing the force and displacement components in the local x, y directions and M_i and θ_i are the moment and rotation (positive in anticlockwise direction) at the beginning of the element (e). The axial behaviour of the element is can represent by the equation 3.4 for the element end acting force and resulting displacement relation is in simplicity,

$$R_{xi} = -R_{xj} = \left(\frac{EA}{L}\right)^e (\delta_{xi}^e - \delta_{xj}^e) \qquad (3.4)$$

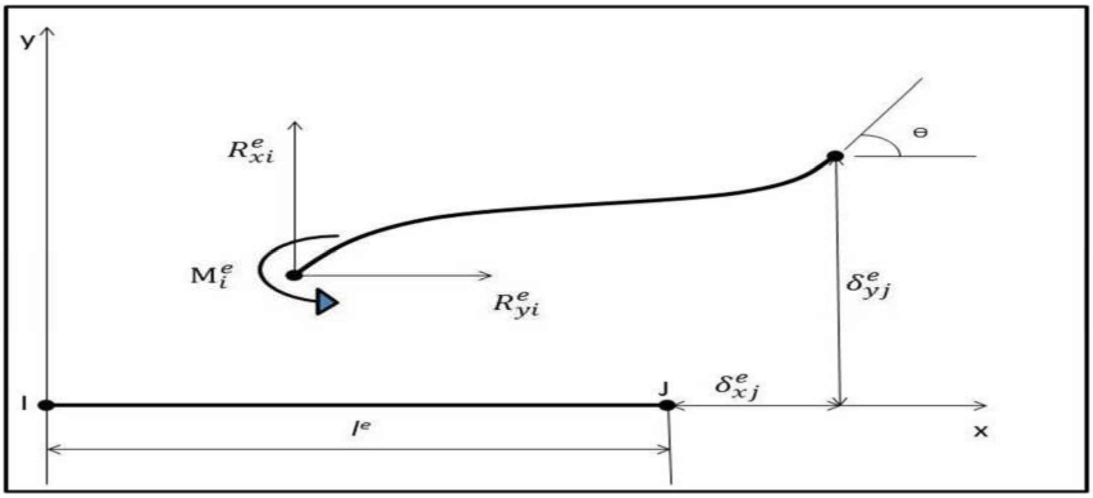


Figure 3.2 2D element local coordinate.

Where E denoted by the elastic modulus of the material and A represents the element cross section area which is constant along its length. The moment at the ends of the element are driven by the method of slope deflection equation,

$$M_i=2k^e(2\theta_i^e+\theta_j^e+\frac{3\left(\delta_{yi}^e-\delta_{yj}^e\right)}{L})$$
$$M_j=2k^e(2\theta_j^e+\theta_i^e+\frac{3\left(\delta_{yi}^e-\delta_{yj}^e\right)}{L}) \quad (3.5)$$

From equilibrium by taking a moment about the two ends of the element;

$$R_{yi}^e = -R_{yj}^e = (M_i^e + M_j^e)/L$$
$$= \frac{12EI}{L^3}\left(\delta_{yi}^e - \delta_{yj}^e\right) + \frac{6EI}{L^2}(\theta_i^e - \theta_j^e) \quad (3.6)$$

From the above equation we can assembly in on matrix form of the format shown below

$$\begin{Bmatrix} R_i^e \\ R_j^e \end{Bmatrix} = \begin{bmatrix} K_{ii}^e & K_{ij}^e \\ K_{ji}^e & K_{jj}^e \end{bmatrix} \begin{Bmatrix} \delta_i^e \\ \delta_j^e \end{Bmatrix} \quad (3.7)$$

Equation 3.8 displays the stiffness matrix and the DOFs for a 2D frame element.

$$\begin{Bmatrix} R^e_{xi} \\ R^e_{yi} \\ m^e_i \\ R^e_{xj} \\ R^e_{yj} \\ m^e_j \end{Bmatrix} = \begin{bmatrix} \frac{AE}{L} & 0 & 0 & \frac{-AE}{L} & 0 & 0 \\ 0 & \frac{12EI}{L^3} & \frac{6EI}{L^2} & 0 & \frac{-12EI}{L^3} & \frac{6EI}{L^2} \\ 0 & \frac{6EI}{L^2} & \frac{4EI}{L} & 0 & \frac{-6EI}{L^2} & \frac{4EI}{L} \\ \frac{-AE}{L} & 0 & 0 & \frac{AE}{L} & 0 & 0 \\ 0 & \frac{-12EI}{L^3} & \frac{-6EI}{L^2} & 0 & \frac{12EI}{L^3} & \frac{-6EI}{L^2} \\ 0 & \frac{6EI}{L^2} & \frac{2EI}{L} & 0 & \frac{-6EI}{L^2} & \frac{4EI}{L} \end{bmatrix} \begin{Bmatrix} u_{xi} \\ u_{yi} \\ \theta^e_i \\ u_{xj} \\ u_{yj} \\ \theta^e_j \end{Bmatrix} \tag{3.8}$$

The process by which can be transform these equations to the global coordinates system X, Y for simply an individual element was described in the particular form below:

$$[K^e_{ij}] = [T][K'_{ij}][T]^T \tag{3.9}$$

The transformation matrix T which it is the relation between force and displacement components in the local and global coordinate systems can be written,

$$[T] = \begin{bmatrix} \text{Cos}\alpha & -\text{Sin}\alpha & 0 \\ \text{Sin}\alpha & \text{Cos}\alpha & 0 \\ 0 & 0 & 1 \end{bmatrix} \tag{3.10}$$

3.2.1.1 Three dimensional frame element

For an element subject to bending moments, a 3D frame element is used. Dealing with a local coordinate system with six degrees of freedom per node as shown in the Figure 3.3 below:

When axial force is applied to the frame element (DOFs 1 and 7), bending in the local 1-2 plane (DOFs 2, 6, 8, and 12), bending in the local 1-3 plane (DOFs 3, 5, 9 and 11), and for applied torsion (DOFs 4 and 10).

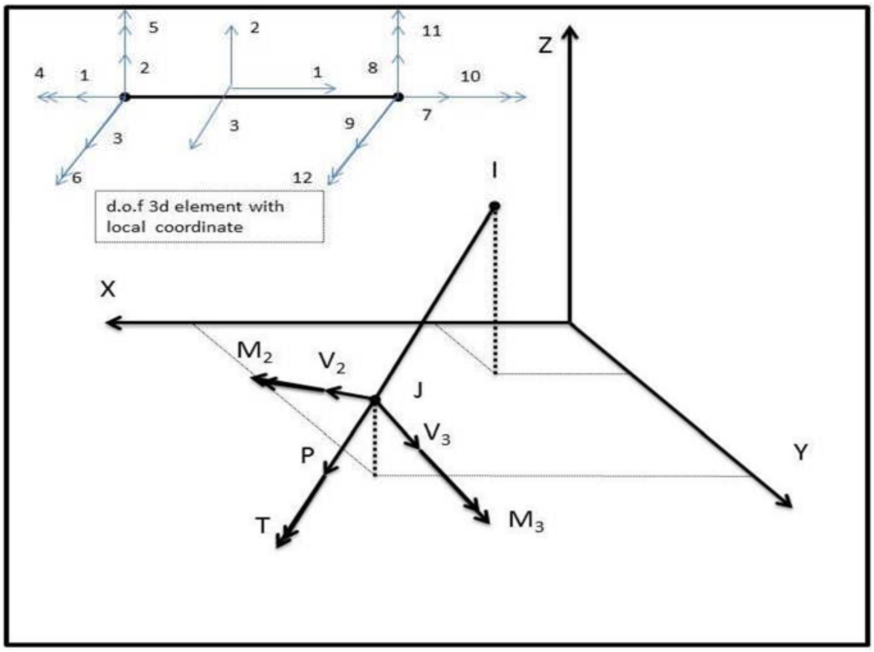


Figure 3.3 3D element subject to bending.

The section properties, element length, and material properties were represented in the stiffness coefficients. When in the case of axial stiffness the functions have represented the element length, L, modulus of elasticity, E, and the cross sectional area, A. Consequently, for bending in the local 1-2 plane are moment of inertia for bending about the local 3-axis, I_3, E, and L. While for bending in the local 1-3 plane are moment of inertia for bending about the local 2-axis, I_2, E, and L. Finally for torsion are the length, L, the shear modulus, G, and the torsional constant, J [40].

Where the principal moment of inertia was represented by I_2 and I_3 with the central axis of the element is run through the shear center. Besides, the classic beam theory is the basis of the element, with the consideration that warping of the cross section does not exist. For symmetric, solid, or closed sections, warping is either nonexistent or unimportant and the shear center is at the centroid of the section [39].

The evaluation of the frame element in 3D stiffness as mentioned before represents by a regular expanding of the equations described in the 2D element. Shearing and bending deformations can be involved in the normal direction using the same equations. Furthermore, it is clear that the isolated torsional flexibility is given in equation (3.36) below [43],

$$F_T = \sum_{i}^{Imax} \int_{s_i}^{s_{i+1}} \frac{1}{G(s)J(s)} ds \qquad (3.11)$$

While for the term of torsional stiffness, G (s), and polar moment of inertia, J (s), it could be difficult to calculate for several cross-sections. The application of the FE mesh might be necessary with complex sections. In which the basis of the evolution of the beam element by using FEM. The stiffness matrix of any type of element is obtained from strain energy, hence the stiffness matrix for the case of bending in any plane it can be written [43],

$$k_b' = \int_0^L [B]^T \cdot EI \cdot [B] \cdot dx \qquad (3.12)$$

Where [B] is based on a supposed group of shape functions and give curvature at any specified point (x) as a function of nodal displacement. In the same manner we can describe the stiffness for the axial load element which is represented by,

$$k_a' = \int_0^L [B]^T \cdot EA \cdot [B] \cdot dx \qquad (3.13)$$

Where [B] here is provides axial stretch along the length of the element as a function of nodal displacement, and further more the derivation for torsion element is;

$$k_T' = \int_0^L [B]^T \cdot GJ \cdot [B] \cdot dx \qquad (3.14)$$

In this case [B] provides the twist along the length of the element as a function of nodal displacements. Opposed to prismatic element for tapered in which there is variation in moment of Inertia (I), cross sectional Area (A) and polar moment of inertia (J), hence the variation can include naturally as a part of integrals. We can compute the stiffness coefficients by defining shape functions, for developing the [B] matrix and integrating numerically for each of the $[k_b^{'}]$, $[k_a^{'}]$and $[k_T^{'}]$.

For the stiffness matrix $[k_b^{'}]$ due to bending is obtained by subdividing the element longitudinally the simplest Bernoulli Euler beam element which has to node, i and j, and four degrees of freedom shown in the node displacement vector [44];

$$[u]^e = \begin{bmatrix} u_i^e & \theta_i^e & u_j^e & \theta_j^e \end{bmatrix}^T \tag{3.15}$$

In this case for C(1) continuity which is the simplest shape function meet this continuity for the nodal freedom configuration by choosing polynomial interpolation function called Hermitian cubic shape functions, hence the interpolation formula of the functions can be written as [44]

$$[N_1^e \quad N_2^e \quad N_3^e \quad N_4^e]\begin{bmatrix} u_i^e \\ \theta_i^e \\ u_j^e \\ \theta_j^e \end{bmatrix} = Nu^e \tag{3.16}$$

Here it is assumed the natural coordinate ξ, which varies from -1 when (x=0) to +1 at (x=L) along the length of the member, while ξ= (2x/L) -2 then the shape function for bending with the 1-2 plane is calculated as [44]. It is shown in Figure 3.4.

$$N_1^e = \frac{1}{4}(1-\xi)^2(2+\xi), \qquad N_3^e = \frac{1}{4}(1+\xi)^2(2-\xi),$$
$$N_2^e = \frac{1}{8}L(1-\xi)^2(1+\xi), \qquad N_4^e = -\frac{1}{8}L(1+\xi)^2(1-\xi) \tag{3.17}$$

In the equation of total potential energy

$$\Pi = U - W \tag{3.18}$$

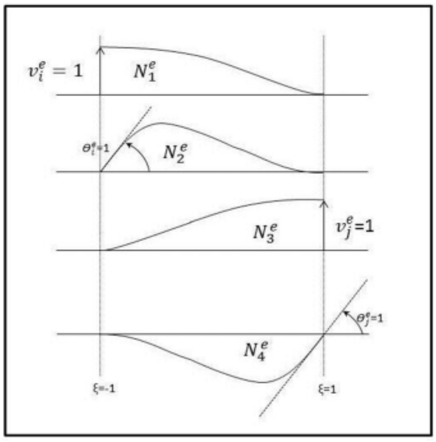


Figure 3.4 Cubic shape functions.

Where U and W denote the internal and external energies and for U are including only bending in Bernoulli-Euler model [44],

$$U=\frac{1}{2}\int_V \sigma\epsilon dV = \frac{1}{2}\int_0^L Mkdx = \frac{1}{2}\int_0^L EIk^2dx = \frac{1}{2}\int_0^L EI(v'')^2dx \quad (3.19)$$

$$W=\int_0^L qvdx$$

Where the q is the applied force and the curvature k in U can be expressed in the terms of differentiating twice the nodal displacements with respect to *x*:

$$k=\frac{d^2v^e(x)}{dx^2}=\frac{4}{L^2}\frac{d^2v^e(\xi)}{d\xi^2}=\frac{4}{L^2}\frac{dN^e}{d\xi^2}u^e=[B][u]^e=N''[u]^e \quad (3.20)$$

For this [B]= N" and it is 1×4 curvature displacement matrix as shown:

$$[B]=\frac{1}{L}[6\frac{\xi}{L} \quad 3\xi-1 \quad -6\frac{\xi}{L} \quad 3\xi+1] \quad (3.21)$$

Transforming of integrating the natural coordinate system obtaining the following integral from the mentioned integral before,

$$[k_b'] = \int_0^L [\mathrm{B}]^{\mathrm{T}}.\,\mathrm{EI}.\,[\mathrm{B}].\,\mathrm{dx} = \int_{-1}^{1} \mathrm{EI}.\,[\mathrm{B}]^{\mathrm{T}}.\,[\mathrm{B}].\frac{1}{2}.\,\mathrm{L}.\,\mathrm{d}\xi \qquad (3.22)$$

When substituting the above integral with value of [B] and $[\mathrm{B}]^{\mathrm{T}}$ from (3.22) to get the following integral as a function of (ξ):

$$[k_b'] = \frac{\mathrm{EI}}{2\mathrm{L}}\int_{-1}^{1}\begin{bmatrix} 6\frac{\xi}{\mathrm{L}} \\ 3\xi-1 \\ -6\frac{\xi}{\mathrm{L}} \\ 3\xi+1 \end{bmatrix}\begin{bmatrix} 6\frac{\xi}{\mathrm{L}} & 3\xi-1 & -6\frac{\xi}{\mathrm{L}} & 3\xi+1 \end{bmatrix}\mathrm{d}\xi \qquad (3.23)$$

We get integral of a matrix of 4×4 symmetric matrix from the above multiplication of matrices which is discussed in next sections the operation on matrices with the sections we explained numerical techniques, hence we get the following integral [44],

$$[k_b'] = \frac{\mathrm{EI}}{2\mathrm{L}^3}\int_{-1}^{1}\begin{bmatrix} 36\xi^2 & 6\xi(3\xi-1)\mathrm{L} & -36\xi^2 & 6\xi(3\xi+1)\mathrm{L} \\ & (3\xi-1)^2\mathrm{L}^2 & -6\xi(3\xi-1)\mathrm{L} & (9\xi^2-1)\mathrm{L}^2 \\ & & 36\xi^2 & -6\xi(3\xi+1)\mathrm{L} \\ \mathrm{sym} & & & (3\xi+1)^2\mathrm{L}^2 \end{bmatrix}.\,\mathrm{d}\xi \qquad (3.24)$$

Integration is performed numerically, using n integration point for ξ and weight factors are applied finally we get [44],

$$[k_{b12}'] = \frac{\mathrm{EI}_3}{\mathrm{L}^3}\begin{bmatrix} 12 & 6\mathrm{L} & -12 & 6\mathrm{L} \\ & 4\mathrm{L}^2 & -6\mathrm{L} & 2\mathrm{L}^2 \\ & & 12 & -6\mathrm{L} \\ \mathrm{sym} & & & 4\mathrm{L}^2 \end{bmatrix} \qquad (3.25)$$

This matrix is cover the degrees of freedom 2, 6, 8 and 12 shown in Figure 3.3 similarly for bending with the 1-3 plane to pertains the degrees of freedom 3,5,9 and 11. $[k_a']$ was mentioned before as the stiffness matrix for axial force is derived similarly from the linear shape functions as ξ=x/L [39,42]

$$[N] = [1-\xi \quad \xi] \tag{3.26}$$

The derived strain-displacement matrix is calculated and the result shown below:

$$[B]=\begin{bmatrix}-\frac{1}{L} & \frac{1}{L}\end{bmatrix} \tag{3.27}$$

Substituting [B] and $[B]^T$ in equation (3.13) and plugging in and multiplying to get:

$$[k_a'] = \frac{AE}{L}\int_{-1}^{1}\begin{bmatrix}1 & -1\\ -1 & 1\end{bmatrix}.d\xi \tag{3.28}$$

Finally for a stiffness matrix in the torsion case which pertains the degrees of freedom 4 and 10 in Figure 3.3 the using of exact same approach and it is find that:

$$[k_T']=\frac{JG}{L}\int_{-1}^{1}\begin{bmatrix}1 & -1\\ -1 & 1\end{bmatrix}.d\xi \tag{3.29}$$

Numerical integrals are used for the stiffness contributions in the case of torsion, bending, and axial force before collective in the model of the total stiffness matrix of the element in the local axis of the system. The direction of the local axis and the transformation matrix are applied to get the final total stiffness matrix for prismatic element.

3.2.2 Geometric stiffness matrix

The geometric stiffness matrix for 3D element is calculated from the basics of virtual work. This study is explained in the a textbook by Yang and Kuo (1994) [45]. This matrix is written in terms of the present element forces, cross section area, moment of inertia, member length and torsional constant.

$$\begin{gathered}P_{avg.}=\frac{1}{2}\left(P_j\text{-}P_i\right),\ a=\frac{P_{avg.}}{L},\ b=1.2a+\frac{12P_{avg.}I_3}{AL^3},\\ c=1.2a+\frac{12P_{avg.}I_2}{AL^3},\ d=\frac{M_{2i}}{L},\ e=\frac{M_{3i}}{L},\end{gathered} \tag{3.30}$$

$$f=\frac{P_{avg.}J}{AL},\ g=\frac{T_j}{L},\ h=\frac{P_{avg.}}{10}+\frac{6P_{avg.}I_2}{AL^2},\ i=\frac{M_{3i}+M_{3j}}{6},$$

$$j=\frac{2P_{avg.}L}{15}+\frac{4P_{avg.}I_2}{AL},\ k=\frac{P_{avg.}}{10}+\frac{6P_{avg.}I_3}{AL^2},l=-\frac{M_{2i}+M_{2j}}{6}$$

$$,\ m=\frac{2P_{avg.}L}{15}+\frac{4P_{avg.}I_3}{AL},\ n=\frac{M_{2j}}{L},\ o=\frac{M_{3j}}{L},$$

$$p=-\frac{P_{avg.}L}{30}+\frac{2P_{avg.}I_2}{AL},\ q=-\frac{T_j}{2},\ r=-\frac{P_{avg.}I_3}{30}+\frac{2P_{avg.}I_3}{AL}$$

These coefficients are used in the 12×12 matrix as represent in

$$[k_g]=\begin{bmatrix} a & 0 & 0 & 0 & -d & -e & -a & 0 & 0 & 0 & -n & -o \\ & b & 0 & d & g & k & 0 & -b & 0 & n & -g & k \\ & & c & e & -h & g & 0 & 0 & -c & o & -h & -g \\ & & & f & i & l & 0 & -d & -e & -f & -i & -l \\ & & & & j & 0 & d & -g & h & -i & p & -q \\ & & & & & m & e & -k & -g & -l & q & r \\ & & & & & & a & 0 & 0 & 0 & n & o \\ & & & & & & & b & 0 & -n & g & -k \\ & & & & & & & & c & -o & h & g \\ & \text{symm} & & & & & & & & f & i & l \\ & & & & & & & & & & j & 0 \\ & & & & & & & & & & & m \end{bmatrix} \qquad (3.31)$$

The geometric stiffness matrix $[k_g]$ which is derived by Yang and Kuo (1994) [45] for the 3D element has been said to consist of bending moments of the quasi tangential type and torsional moments of the semi tangential type, the two of them was defined as stress resultant of the member cross sections. The derived $[k_g]$ represents only part of the terms dealing with the rotational properties of nodal moments. In addition, Yang and Kuo (1994) proposed a modification that results from the effect of point rotation on torque. The prepared stiffness matrix is called the joint moment matrix $[k_j]$ with dimensions 12×12:

$$[k_j]=\begin{bmatrix}[0] & & & \\ & [s_i] & & \\ & & [0] & \\ & & & [s_j]\end{bmatrix},\ [s_i]=\begin{bmatrix}0 & -\frac{M_{3i}}{2} & \frac{M_{2i}}{2} \\ -\frac{M_{3i}}{2} & 0 & 0 \\ \frac{M_{2i}}{2} & 0 & 0\end{bmatrix} \tag{3.32}$$

Finally the complete tangent stiffness matrix with the local system can be calculated and obtained from:

$$\left[k_t^{'}\right] = [k] + \left[k_g\right] + [k_j] \tag{3.33}$$

Transforming the above equation to global form by;

$$[k_t^e]=[T][k_t^{'}][T]^T \tag{3.34}$$

3.2.3 Fixed end forces

Fixed element loads for element in a frame element in 3D comes from the weight of element, wind, changing in temperature and distributed ice load. The fixed end loads were given in simple equations in many texts prepared for prismatic element only one point of view is taken that the load components given is in the local frame coordinates. In short mean they must be transformed to the global coordinates of the system.

Member weight: (γ_x,γ_y,γ_z) the global components of material weight density are given. The calculation of global components of distributed force is obtained by multiplying the cross sectional area. In the direction of local force components along the differential length derived by transforming into local components:

$$\begin{Bmatrix} w_{x2}^{'} \\ w_{y2}^{'} \\ w_{z2}^{'} \end{Bmatrix} = A.\begin{bmatrix} l & m & n \\ l_2 & m_2 & n_2 \\ l_3 & m_3 & n_3 \end{bmatrix}\begin{Bmatrix} \gamma_x \\ \gamma_y \\ \gamma_z \end{Bmatrix} \tag{3.35}$$

When the member weight is distributed load on the length of the member and can write in the term of FE on the basis of virtual work as driving below: [44]

$$f^e = \frac{1}{2} wL \int_{-1}^{1} [N]^T d\xi$$

$$= \frac{1}{2} wL \int_{-1}^{1} \begin{bmatrix} \frac{1}{4}(1-\xi)^2(2+\xi) \\ \frac{L}{8}(1-\xi)^2(1+\xi) \\ \frac{1}{4}(1-\xi)^2(2-\xi) \\ -\frac{L}{8}(1-\xi)^2(1-\xi) \end{bmatrix} d\xi = wL \begin{bmatrix} \frac{1}{2} \\ \frac{L}{12} \\ \frac{1}{2} \\ -\frac{L}{12} \end{bmatrix} \quad (3.36)$$

This displays that a uniform load w over the beam element in one plane maps to two transverse node loads wL/2, as may be expected, plus two nodal moments ± $wL^2/12$. The last is called the fixed-end moments in the FEM literature. Then, the fixed end moment vector in the local system is:

$$\{f\}^{e'} = \left\{ -\frac{w'_{x2}L}{2} \quad -\frac{w'_{y2}L}{2} \quad -\frac{w'_{z2}L}{2} \quad 0 \quad -\frac{w'_{z2}L^2}{12} \quad -\frac{w'_{y2}L^2}{12} \right.$$
$$\left. -\frac{w'_{x2}L}{2} \quad -\frac{w'_{y2}L}{2} \quad -\frac{w'_{z2}L}{2} \quad 0 \quad -\frac{w'_{z2}L^2}{12} \quad -\frac{w'_{y2}L^2}{12} \right\}^T \quad (3.37)$$

3.2.4 Transformation from local to global system

The expression [T] is denoted to transformation matrix for the elements of space frames can be obtained using the principals of trigonometric direction cosines with respect all three (x, y and z) axes of the member local coordinate system with respect to the structure’s global axis (X,Y and Z) coordinate system.

The algebraic sums of the components of the global forces F_1,F_2 and F_3 in the directions of the local coordinates x, y and z axis which is shown in the Figure 3.5 it can be calculated as:

$$F_1 = F_1' \cos\theta_{xX} + F_2' \cos\Theta_{xY} + F_3' \cos\Theta_{xZ}$$
$$F_2 = F_1' \cos\Theta_{yX} + F_2' \cos\Theta_{yY} + F_3' \cos\Theta_{yZ} \qquad (3.38)$$
$$F_3 = F_1' \cos\Theta_{zX} + F_2' \cos\Theta_{zY} + F_3' \cos\Theta_{zZ}$$

This equation can be written in the Matrix form:

$$\begin{Bmatrix} F_1 \\ F_2 \\ F_3 \end{Bmatrix} = \begin{bmatrix} \cos\Theta_{xX} & \cos\Theta_{xY} & \cos\theta_{xZ} \\ \cos\Theta_{yX} & \cos\Theta_{yY} & \cos\Theta_{yZ} \\ \cos\Theta_{zX} & \cos\Theta_{zY} & \cos\Theta_{zZ} \end{bmatrix} \begin{Bmatrix} F_1' \\ F_1' \\ F_1' \end{Bmatrix}, \qquad (3.39)$$

$$r = \begin{bmatrix} \cos\Theta_{xX} & \cos\Theta_{xY} & \cos\Theta_{xZ} \\ \cos\Theta_{yX} & \cos\Theta_{yY} & \cos\Theta_{yZ} \\ \cos\Theta_{zX} & \cos\Theta_{zY} & \cos\Theta_{zZ} \end{bmatrix}$$

This matrix represents one node end force transformation from local to global system of element and we do this operation for other end forces and the final matrix form can write in the 12×12 matrix as collected below:

$$T = \begin{bmatrix} r & 0 & 0 & 0 \\ 0 & r & 0 & 0 \\ 0 & 0 & r & 0 \\ 0 & 0 & 0 & r \end{bmatrix} \qquad (3.40)$$

3.3 The Numerical Basis in Solving Analysis Matrix

When we analyze structures with the stiffness matrix method it is required to know some basic on numerical analysis to get results. Today with high motivation computers we can do these operations preciously and quickly the important of this work is that you have experience to obtain accurate results with minimum steps to saving time.

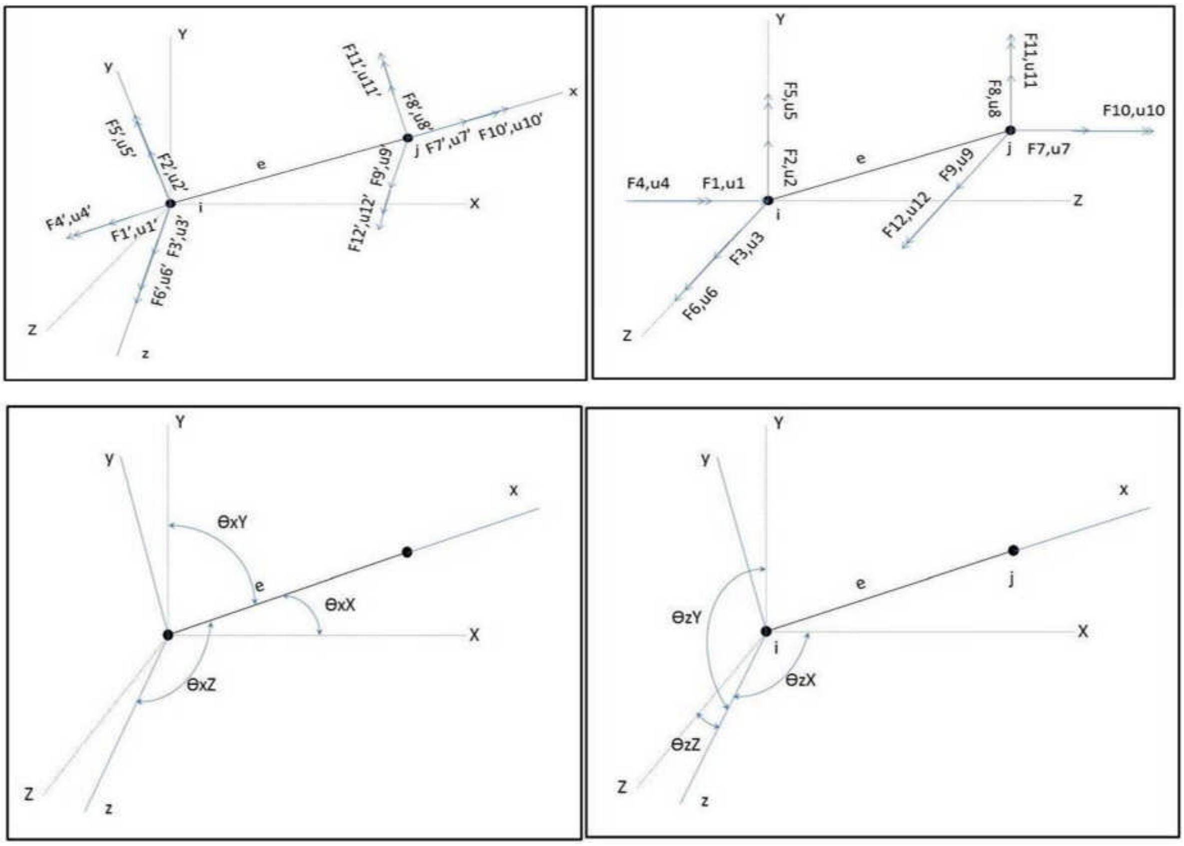


Figure 3.5 Global and local forces and displacement for 3D member frame.

3.3.1 Operation on matrix

There are many operations work with matrices and many texts on numerical analysis explained these operations in this section we show only the multiplication operation which is important and give the final size of the stiffness matrix. Multiplication of two matrices can be performed only if the columns number of the first matrix should be equal to the rows number of the second matrix. Such matrices are said to be conformable for multiplication [41].

$$[A]_{(l,m)}\,[B]_{(m,n)} = [C]_{(l,n)} \tag{3.41}$$

3.3.2 Gause Jordan elimination method

The Gauss Jordan elimination method is one of the most used procedures for solving simultaneous linear equations and for determining inverses of matrices [41].

3.3.3 Numerical integration techniques

The integrals that represent the element matrices are written with respect to dx and we must make a change in the integration variables to transfer the integral with the evaluated natural coordinate system [46].

$$\int_{X_i}^{X_m} f(x)dx = \int_{p_1}^{p_2} g(p)(\frac{d(x(p))}{dp})dp \qquad (3.42)$$

Where x(p) is the equation that have relation of the two coordinates system in which the term:

$$\frac{d(x(p))}{dp} = [J] \qquad (3.43)$$

Is called the jacobian matrix for the transformation equation. We evaluate the integral using the natural coordinate ξ therefore the equation (3.41) becomes:

$$\int_{X_i}^{X_m} f(x)dx = \int_{-1}^{1} g(\xi)(\frac{d(x(\xi))}{d\xi})d\xi \qquad (3.44)$$

We must have an equation that gives the relation of x and ξ. This transformation is written using the element shape function and the global coordinates of the nodes. Here we can deal with numerical integration techniques associated with the natural coordinates ξ relative to evaluate the matrices of element which is the Gause Legendre quadrature. This locates the sampling points to achieve the maximum accuracy. In the short manner this mean if take n sampling points a polynomial of (2n-1) can be integrated exactly. The sampling points are in the interval -1 to +1 showing the location of points for n=2 and n=3 are shown in Table 3.1 below [46].

The result of our examples showed in the end of this chapter is solved by using two point 3 Gause polynomial degree to get the result of analysis shown in the tables for deflections.

Table 3.1 Location and weights for Gause- Legendere Quadrature.

n	ξ_i	W_i
1	0.0	2.0
2	±0.577350	1.0
3	0.0 ±0.774597	8/9 5/9
4	±0.861136 ±0.339981	0.347855 0.652145

3.4 Loadings and Loading Cases on Transmission Tower

Transmission line for design purpose must include loadings from many sources. Transmission line tower loading is involve of three commonly perpendicular systems of loads working normal, vertically and parallel to the line direction [3].

3.4.1 Transverse load

The loads were carried by the ground wire spport and points of conductor with the direction perpendicular to the line way as shown in Figure 3.6 including the wind load distributed over the transverse face of the structure of the tower.

3.4.2 Longitudinal load

This loads acting in the direction parallel to the line on the tower structure and is happening with unequal conductor tension to the tower structure.

3.4.3 Vertical load

This type of load is applied to the ends of the cross arms and on the peak point of ground wire downward and consist of the following vertical components:

- Weight of conductor itself and the weight of the ice if included in the covering weight span.
- Weight of insulators and accessories.
- Extra loads to be include for the weight of the man with maintenance tools.

3.4.4 Weight of the structure

The weight of the structure is unknown till the complete design of the structure is complete. First we put the initial weight of the structure by assuming reasonable cross section area and geometry of the tower and checking for design with the requirement of the codes and stress constraints.

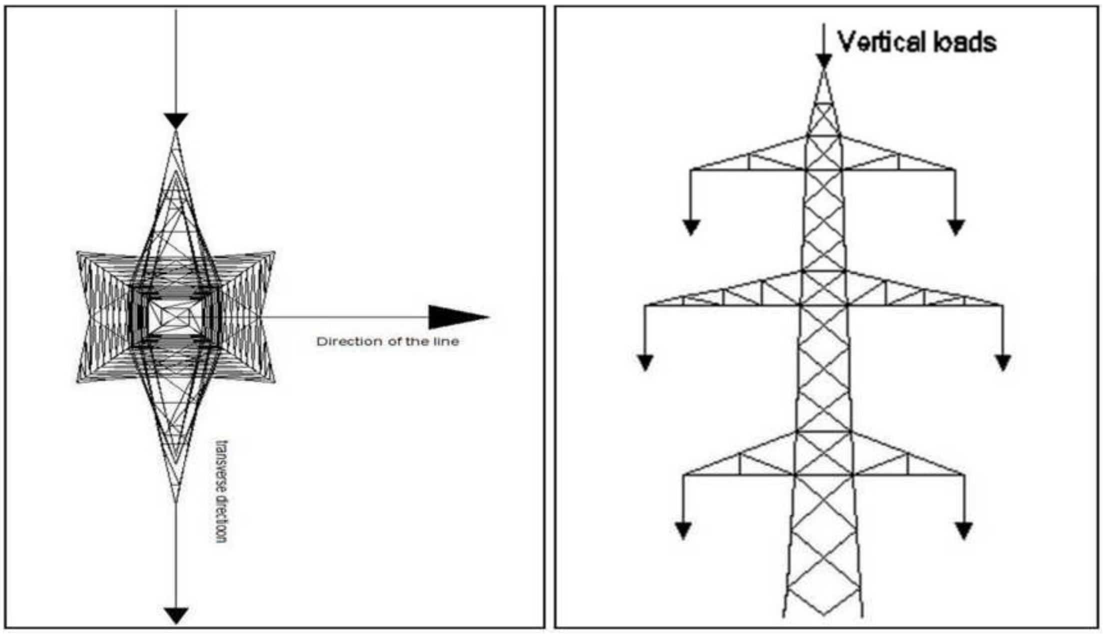


Figure 3.6 Direction of loading for transmission tower.

3.5 Examples of Three Dimension Frame Analysis

To perform that the previous formulation of the stiffness matrix is work which are coded with FORTRAN starting with simple frame examples extended to more complicated example for transmission tower which has relation with the study for static analysis and comparing results with program SAP2000 for checking the results or with literature if available.

3.5.1 Three dimension four bar frame

Problem definition: The four bar frame of Figure 3.7 is to be analyzed. Material properties are: E=0.2×10^9 kN/m^2 and material density ρ=1.0 kg/m^3 and Poisson's ratio=0.3. Two horizontal loads of (2.0 kN) and (-0.4 kN) are imposed in the x and y-direction at node 5, along with (3.0 kN) upward vertical load. The cross sectional areas of members are 0.001 m^2.

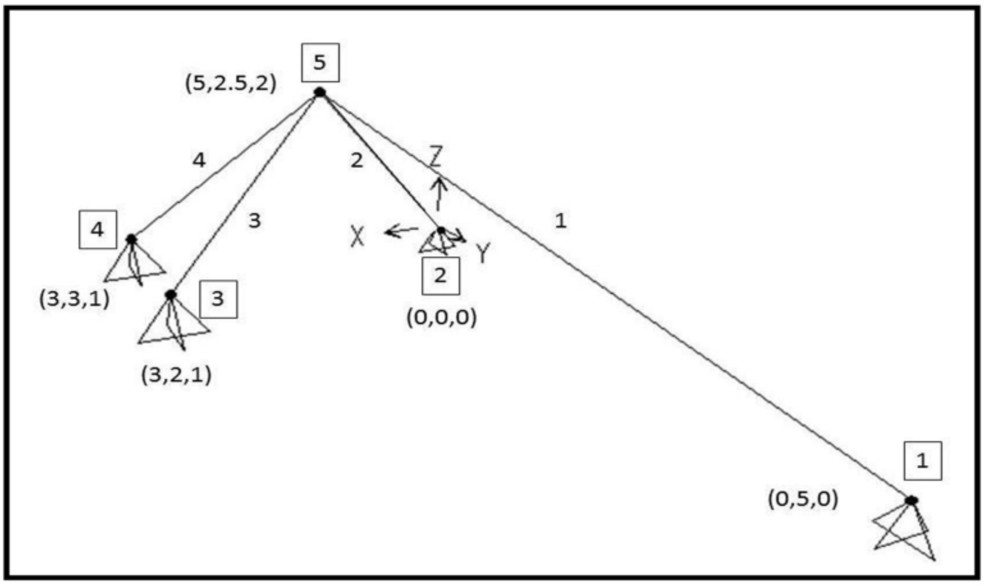


Figure 3.7 Node coordinates and element number for 3D four bar frame.

Discussions of results: The analysis by the FEM coded program is performed and the analysis result with three nodes and two gause quadrature is obtained. These results are compared with source program (SAP2000) to show the efficiency of the program. The results are tabulated in Table 3.2 for comparison of 4bar results and from the table the results is can be said close to each other.

Table 3.2 Comparisons of displacement at point 5 for four bar 3D frame including self weight of structure.

Results	U_1 (m)	U_2 (m)	U_3 (m)	Weight (Kg)
Present	-0.398378×10^{-4}	-0.52558×10^{-4}	0.440217×10^{-4}	0.010553
SAP2000	-0.3941×10^{-4}	-0.5255×10^{-4}	0.4335×10^{-4}	0.01055

3.5.2 Three dimensional twenty five bar truss

Problem definition: The 25-bar 3D frame of Figure 3.8 is to be analyzed by frame program. Material properties are: Young's modulus, E = 1.0×10^4 ksi, material density, ρ = 0.1 lb/in^3, Nodes 7, 8, 9 and 10 are fully constrained and nodes 1, 2, 3 and 6 are loaded with different load values see Table 3.3. Cross-sectional areas of members are 3.0 in^2 [48, 49].

Table 3.3 Loading for 25 bars 3D truss.

Nodes	P_x (lb)	P_y (lb)	P_z (lb)
1	1000	-10000	-10000
2	0	-10000	-10000
3	500	0	0
6	600	0	0

Discussion of Results: The analysis by the FEM coded program is performed and the analysis result with three nodes and two gause quadrature is obtained. These results are compared with source program (SAP2000) to show the efficiency of the program. 25 Bar truss results are tabulated in Table 3.4 and compared and there is small difference in weight because the approximation of the gravity in SAP2000.

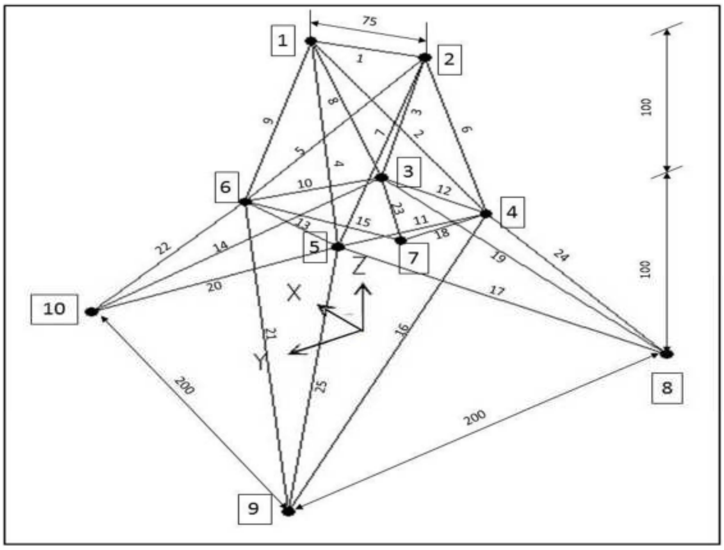


Figure 3.8 25 bar 3D truss element number and dimensions.

Table 3.4 Comparisons of analysis results for 3D 25 bar truss under self weight of the structure.

Joints	Results	U_1 (in)	U_2 (in)	U_3 (in)	weight (lb)
2	Present	0.0120077	-0.259052	-0.0330087	992.474
	SAP2000	0.012	-0.259	-0.033	
1	Present	0.0168350	-0.25875	-0.0407211	992.16
	SAP2000	0.0168	-0.2587	-0.0407	

3.5.3 Three dimensions (672) bars and (306) nodes transmission tower

Problem definition: A transmission tower loaded due to earth wire and conductor loading as shown in the Figure 3.9 at nodes (a, b, c and d) and have material property for steel E=200×10^9 N/m^2 and ρ=7860 Kg/m^3 and Poisson's ratio=0.3 and bottom nodes are fully constrained.

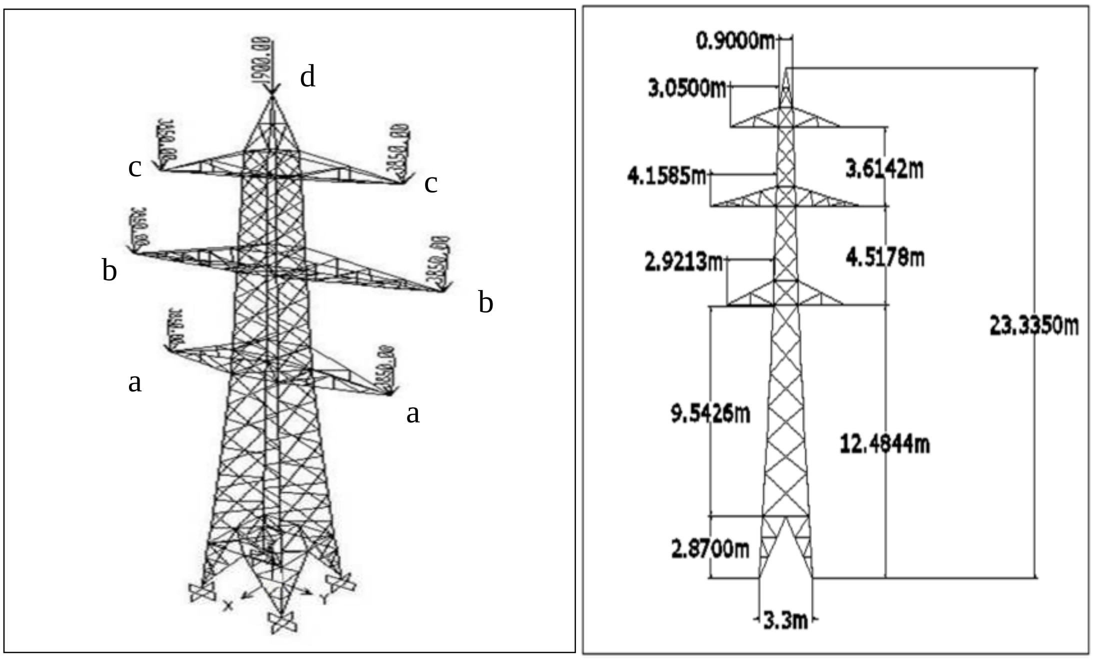


Figure 3.9 Transmission tower, dimensions (m) and applied loads (N).

Discussion of Results: The analysis by the FEM coded program is performed and the analysis result with three nodes and two gause quadrature is obtained. These results are compared with source program (SAP2000) to show the efficiency of the program. The results are tabulated in Table 3.5 and specified nodes shown in Figure 3.9 for comparison of the tower results.

Table 3.5 Comparisons of deflection at top and end of crossarms for the transmission tower.

Nodes	Present U_3 (m)	SAP2000 U_3 (m)
a	-1.59×10^{-4}	-1.58×10^{-4}
b	-3.29×10^{-4}	-3.29×10^{-4}
c	-2.08×10^{-4}	-2.08×10^{-4}
d	-1.23×10^{-4}	-1.23×10^{-4}
Total weight (Kg)	48608	48606

3.6 Summary and Conclusions

In this chapter the concept of FE stiffness matrix analysis explained, besides some techniques in numerical analysis used in the FORTRAN program. To confirm the efficiency of the program, results were compared with SAP2000 program. From the comparisons, it is clear that there is a slight difference between the results in the main direction (in direction of applied loads) and in the other directions the displacements are almost zero.

CHAPTER 4

OPTIMIZATION ALGORITHM

4.1 Introduction

In structural design it is essential to get an appropriate cross section for the members of structure so that they can carry the imposed loads economically and safely. This may be done by the use of SO processes taking into account the structure elements width and thickness are differing to achieve a particular purpose of weight minimization with considering certain constraints. Such techniques are iterative and include numerous re-analyses before an optimum solution can be attained. SO implements can be developed by the efficient integration of structural geometry definitions, structural analysis, optimum design and mathematical programming methods.

4.2 Mathematical Definition of Optimization Problem

The variable and objective function explained in the next sections always exist with a optimization problem of the structure:

• **Objective function:** Refers to a function done to sort designs. Any design variable with feasibility in solution, F represents a number which shows the goodness of the design. Generally F chooses in away a minimum value which is better than a maximum one (a minimization problem). Regularly F indicates the weight, effective stress, displacement in a given direction or even the cost of manufacture.

• **Design variable:** Refer to a variables which expresses the design solutions, and could be swapped during the optimization process with upper and lower limit. It represents the selected material or geometry. if it represents geometry, it may interact to a shape sophisticated interpolation or it may simply be the element cross-section, or the sheet thickness [50].

• **Gene:** A gene refers to an arbitrary length represented by bit string. A binary illustration of the bit string is the number of domains from a lower bound. The GA's demonstration of a single factor value for a control factor represents by the gene. Although the control factor must have a lower limit and upper limit, the domain can be separated into several intervals that can be stated by a bit string of the gene's.

• **Individuals:** Refer to a single result in the population. Two forms of results exists for individual groups as shown below:

1. The genotype, refers to the basic genetic data that the GA deals.
2. The phenotype, refers to the illustration of the chromosome in the terms of the model.

• **Population:** A group of individuals refers to population. A number of individuals involving the population of being examined, the individuals and specific information about the search space described the phenotype parameters. The population with two significant aspects applied in GAs are [52].

1. The population size.
2. The initial population generation.

The aim of minimizing the weight of the structure subjecting to constraints for the member stresses or joint displacements is the basis of the optimization process. The expression of mathematical programming of optimization problem was represented in the equations below:

$$\begin{gathered}\text{Minimize } F(x) \\ \text{Subject to } g_i(x) \leq 0;\ i = 1, 2, \ldots n_c\end{gathered} \quad (4.1)$$

Where n_c refers to the number of constraints. While the structure weight minimization is the goal, F(x) is expressed as follows:

$$F(x)=\sum_{j=1}^{nelem.} \rho A_j L_j \qquad (4.2)$$

Where A_j is the j^{th} element cross sectional area, L_j is the j^{th} element length and ρ is the material density. Constraint function $g_i(x)$ can be written in a dimensionless formula as,

$$\left|\frac{\sigma_j}{\sigma_{all.}}\right| -1 \le 0 \qquad (4.3)$$

Where σ_j is the j^{th} member stress and σ_{all} refers to the allowable stress,

$$\left|\frac{d_i}{d_{all.}}\right| -1 \le 0 \qquad (4.4)$$

Where d_i may refer to the displacement u_{xi} and u_{yi} at joint i which is the horizontal and vertical displacement correspondingly the d_{all} is the allowable displacement. The each constraints cannot characterize directly in a design variable expression; hence, they are implicit and their calculation requires using an analysis of truss matrix type [20].

4.3 Types of Structural Optimization Problems

In the most of the optimization texts, x will closely perform several kinds of structural geometric feature. The geometric feature depends on, three classes of optimization problems for structures.

• **Size optimization:** Refer to problems when x is represented thickness of the structural member, (i.e., truss member cross-sectional areas), or a sheet distributed thickness. A truss structure sizing optimization problem is shown in Figure 4.1.

• **Shape optimization:** Refers to problems when x represents the form or shape of several portions of the structural domain of the boundary condition. A set of partial differential equation is describing this condition. An optimal way in the optimization depends on the choosing of the integration domain from the differential equations. State that the structure connectivity is not swapped by shape optimization; forming of new boundaries are not exist.

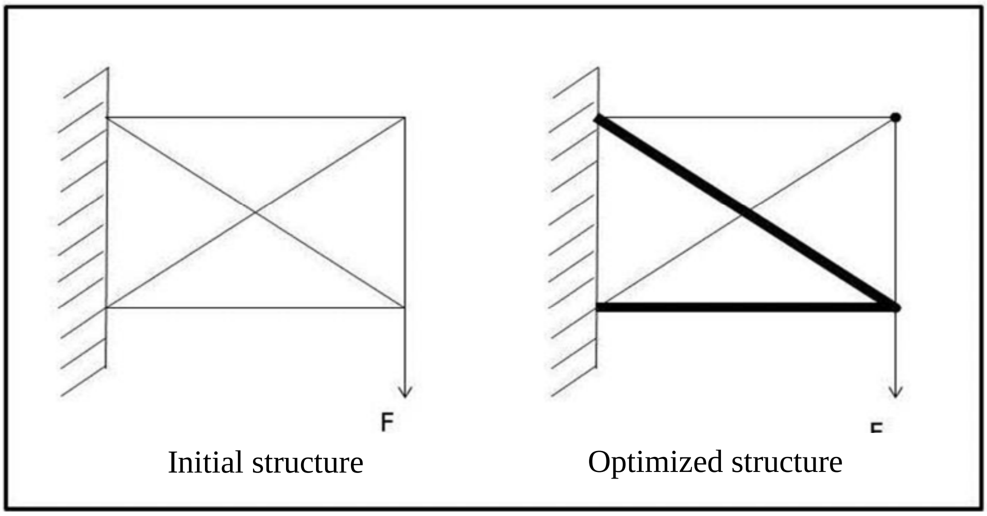


Figure 4.1 A problem of sizing structural optimization (the cross sectional areas is optimized).

• **Topology optimization:** The most general problems of SO are set under this section. In the case of discrete, for example in a truss, optimization process by taking the design variables as truss members cross sectional areas, and so accepting the design variables to choose the zero value, i.e., removed members from the truss. Although the node connectivity is variable, therefore the truss topology is changed, If an alternative of a lattice structure is used imagining of a continuum type structure for example a 2D sheet, when the topology changes allowing the sheet thickness to take the value zero. If optimizing the pure topological features, the optimal thickness must hold only two values: 0 and a fixed maximum sheet thickness. In a 3D case the similar effect can be achieved by allowing x be a density, like a variable that can only take the values 0 and 1 [50].

4.4 Genetic Algorithms for Optimization

GAs are one of the type search optimization that used for minimizing the objective function (the structure weight) and it is different from many algorithms of mathematical programming, is that they do not need the calculation of derivatives of the objective function and the constraints. It's search steps depends on the mechanics of natural genetics and selection. To represent a robust search mechanism it is working on the scheme of the artificial survival of the fittest with genetic operators founds from the nature.

4.4.1 Fundamentals of simple genetic algorithm

Reproduction, crossover, and mutation are the major basics of normal genetics. Their application is used in the genetic search process. GAs advantages compared with other form of the traditional methods of optimization are listed in the following points:

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

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

3. 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.

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

5. During each new production, a creation of new group of strings is produced by the randomized parents from the selection and crossover of the old generation (strings old group). While randomized, GAs as search technique are not simply arbitrary. The efficiencies of the algorithm are exploring the new procedures beside the existing information that obtain a new production with objective function value or improved fitness [51].

4.4.2 Genetic operators

Three genetic operators are included in a simple GA as mentioned before, beginning from a strings initial population (representing possible solutions), the GAs applies the three operators to calculate sequential generations. First, pairs of individuals of the existing population are elected to mate with each other to create the offspring, to examined and then create the generation for next iteration.

4.4.2.1 Selection of parents

In this operator the algorithm elects the chromosome from the population for next operator which is reproduction. 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. After selection of the parent string pair, the application of crossover operator is performed to all of these pairs. Selecting these chromosomes is the problem.

The ability of GAs to obtain nearly optimal or exact optimal solutions under a large selection scheme range. Though, when the selection process is very low, the convergence rate will be slow, and the GA may require unnecessarily longer time to obtain the optimal solution.

Naturally it can recognize two kinds of the selection method, proportionate selection and ordinal-based selection. Selecting individuals built on their fitness values deal with the fitness of the other individuals in the population is the main idea of proportionate based selection. In selection of ordinal-based methods electing individuals not depend on their exact fitness, but depend on their sorting inside the population. This involves that the selection pressure is independent of the fitness distribution of the population, and is only based on the proportional ranking of the population.

It is also probable to apply a scaling function to reorder the fitness range of the population in order to adjust the selection. In another aspect the chance of selecting useful individual in place of poor one will be important. There are several kinds of selection methods clarified below:

- **Roulette wheel selection:** Refers to one of the selection technique of conventional GAs. The application of reproduction operator is usually the operator of proportionate reproductive where choosing a string from the mating pool taking a chance relative to the fitness. The roulette selection basic is a linear search over a roulette wheel with the holes in the wheel weighted in relation to the individual's fitness values. An objective value is put, which is the sum of the fitnesses with a random proportion in the population. The population is moving through up to the objective value is achieved. This is just a moderately robust selection technique, since match individuals are not guaranteed to be elected for, then somewhat have a greater possibility. A match individual will provide more to the target value, but if it does not exceed it, the next chromosome in line has a chance, and it may be poor. It is elementary that the population not be sorted by fitness, because this would dramatically bias the selection. This selection method is simpler to apply but is noisy. The evolution rate depends on the variance of fitness's in the population.

- **Random Selection:** Refers to the method of selecting parents from the population randomly. In terms of disruption of genetic codes, random selection is a little more disruptive, on average that roulette wheel selection [52].

- **Tournament Selection:** A perfect selection approach should be in a way when it has the ability of adjusting selective pressure and population variety so as to fine tune GA search implementation. Disparate to, the selection in Roulette wheel, the selection in tournament approach supplies selective pressure by considering a tournament competition among several individuals. The elite individual from the tournament is the one with the highest fitness, which is the winner of the individuals. Tournament competitions and the winner are then added into the mating pool. The tournament competition is iterated until the mating pool for generating new offspring is filled. The mating pool including of the tournament winner has a higher average population fitness. The fitness difference provides the selection scheme, which makes GA to enhance the fitness of the succeeding genes. This process is more efficient and runs to an optimal solution [52].

- **Boltzmann selection:** The method of simulated annealing is a function of minimization or maximization. These processes simulate the procedure of molten metal slow cooling to attain the minimum function value in a minimization problem. Temperature controlling like parameter established to the theory of Boltzmann probability distribution simulates the cooling phenomenon. This method is not related to this study for this connection the mention of the method is not explained in detail [52].

In the explained section above concluding the mention of fitness of individuals in a GA is the objective function value for its phenotype. In Fitness calculation, the first step is to decode the chromosome and the evaluation of the objective function is to be performed. The fitness not only indicates how good the solution is, but also relates to how close the chromosome is to the optimal one.

4.4.2.2 Recombination or crossover

The crossover operation includes the changing of genetic bit strings (materials) between the two parents. Choosing randomly a place by this operator (along the two chromosomes a bit position) and swaps the sub strings before and after that point between two chromosomes to generate two offspring (new generation). There are various types of crossover can take place, hence these types are of the simplest is explained.

One-point crossover: The easiest kind of the crossover of all of the available kinds. When selecting a pair of individuals to exchange their first few bits and the results are a new pair of children. So suppose a selecting parents pair from the mating pool is displayed in Figure 4.2 below [20],

Parents1	0 0 0 0 1 1 1 1
Parents2	1 1 1 1 0 0 0 0

Figure 4.2 Parents before crossover.

Besides a randomly chosen between 1 and the string length of an integer place n along the string is taken. The established of two new strings by exchanging each character between positions 1 and n inclusively (Goldberg, 1989). Assume in selecting a random number between 1 and 8, taking n = 5, the results of crossover produces the two new strings displayed in Figure 4.3 below,

Parents1	1 1 1 1 0 1 1 1
Parents2	0 0 0 0 1 0 0 0

Figure 4.3 New strings after crossover operation.

The crossover probability (P_c) is an essential parameter in crossover performance. It is a parameter to explain how often crossover will be achieved. The offsprings are exact copies of parents, if there is no crossover. The offsprings create from portions of both parent's chromosomes, if there is crossover. Then all offspring is created by crossover, if the crossover probability is 100%. All new generation is created from exact copies of parent chromosome from old population (but this does not mean that the new generation is the same), When it is 0%. The creation from the crossover is in expect that offsprings will include good parts of the parents and therefore the offsprings will be improved. Though, it is better to allow a specific portion of old population survive to the next generation.

4.4.2.3 Mutation

Every crossover process creates two individuals (children), therefore it will be exposed to the mutation process in the final step to creating the new generation. This process randomly changes or flips values of one or more bit at randomly particular locations in a chromosome bit strings.

The ability of the GA is improved by mutation operator to obtain for a given problem a near optimal solution by providing an adequate level of genetic variation in the population, which is required to make sure that the entire solution space is used in the search for obtaining the best solution. In a sense, it helps as a guarantee; it serves to avoid the loss of genetic material. To illustrate mutation with an example shown in Figure 4.4, assuming that the new parents in the following is obtained from applying one-point crossover [20].

Crossover child 1	1 1 1 1 0 1 1 1
Crossover child 2	0 0 0 0 1 0 0 0

Figure 4.4 Assign mutation point bit string.

Used a random value of x_j which is found between the range:

$$1 \leq x_j \leq lcrom \qquad (4.5)$$

Then starting from the left side, the x_j^{th} bits of the offsprings are swaped from 0 to1 and vice versa. In the Figure 4.4 the x_j=3, then the mutant children are created are explained in Figure 4.5:

Crossover child 1	1 1 0 1 0 1 1 1
Crossover child 2	0 0 1 0 1 0 0 0

Figure 4.5 Offspring created by mutation operation.

The mutation probability (Pm) is the main parameter in the mutation procedure. The adoption of mutation probability is how often portions of chromosome will be mutated. The offspring is created directly after crossover (or directly copied) without modifying, if there is no mutation. When one or more parts of a chromosome are changed, if mutation is applied. If mutation probability is 100%, whole chromosome is changed, if it is 0%, nothing is changed. Mutation usually avoids the GA from dropping into local extremes. Mutation should not occur every time, because then GA will in truth convert to random search.

4.5 Encoding The Variables

A main function of the design variable encoding into chromosomes is that the GA can use them. Subsequently, the chromosomes can be decoded or rated and the evaluation fitness function is performed. Commonly every variable is represented by applying a bit string. every bit string is then combined to make a chromosome, which relates a design variable. There are two types of design variables to be represented by the chromosome.

Discrete variables: When the problems are related with discrete variables, sorting the probable values of whole variables in a table and the bit string works as a pointer specifying the location in the table. Thus a particular bit string is associated with a location in the table and a relating discrete value for the design variable. For example in size optimization ,only a selected set of cross sectional areas is allowed and these are sorted in a table or catalogue. The bit string denotes a position in this table and thus a relating discrete value of the cross sectional area is selected.

Continuous variables (real variables): GAs may approximately characterize continuous or real floating point variables chosen from a given interval. It is essential to split the search range into a number (a two powered) of ranges set by a tolerance (or resolution) specified by the designer. For example in size optimization, visualize that the cross sectional areas interval for every design variable can vary from X_{min} to X_{max}. Let imagining that the range is divided into 1,024 (2^{10}) ranges with 1,024 values recognized in the interval. Therefore the bit string values interval is from 0000000000 to 1111111111 . Furthermore this interval of values needs ten bits. Usually, if lcrom bits are used, then the resolution r is found from the equation [20].

$$r = \frac{X_{max} - X_{min}}{2^{lcrom} - 1} \tag{4.6}$$

4.6 Structure of Genetic Algorithm

The selection, crossover and mutation are three operators to be verified to both simple in computation and effective in attacking a number of important optimization problems. Figure 4.6 demonstrates a chart of the structure for a simple GA. In the starting, the reading of all required data is done and the procedure of the GA will begin for the first generation. The creation of initial population is implemented randomly.

The violation of constraint is calculated and properly modification of the objective function is attained. The fittest design, average, the maximum is obtained, and several convergence criteria, this explained later, are checked. The GA process is terminated, if convergence is achieved; else the GA process returns. Through producing the mating pool, the next population breeding is begun by applying the crossover operator, and the GA process will proceed continuously until convergence, or the maximum number of allowable generations, is completed.

4.7 Constraint in Genetic Algorithm

In unconstrained optimization problems GAs is the best appropriate algorithm. Many engineering problems are constraint optimization, hence to handle the constraint a

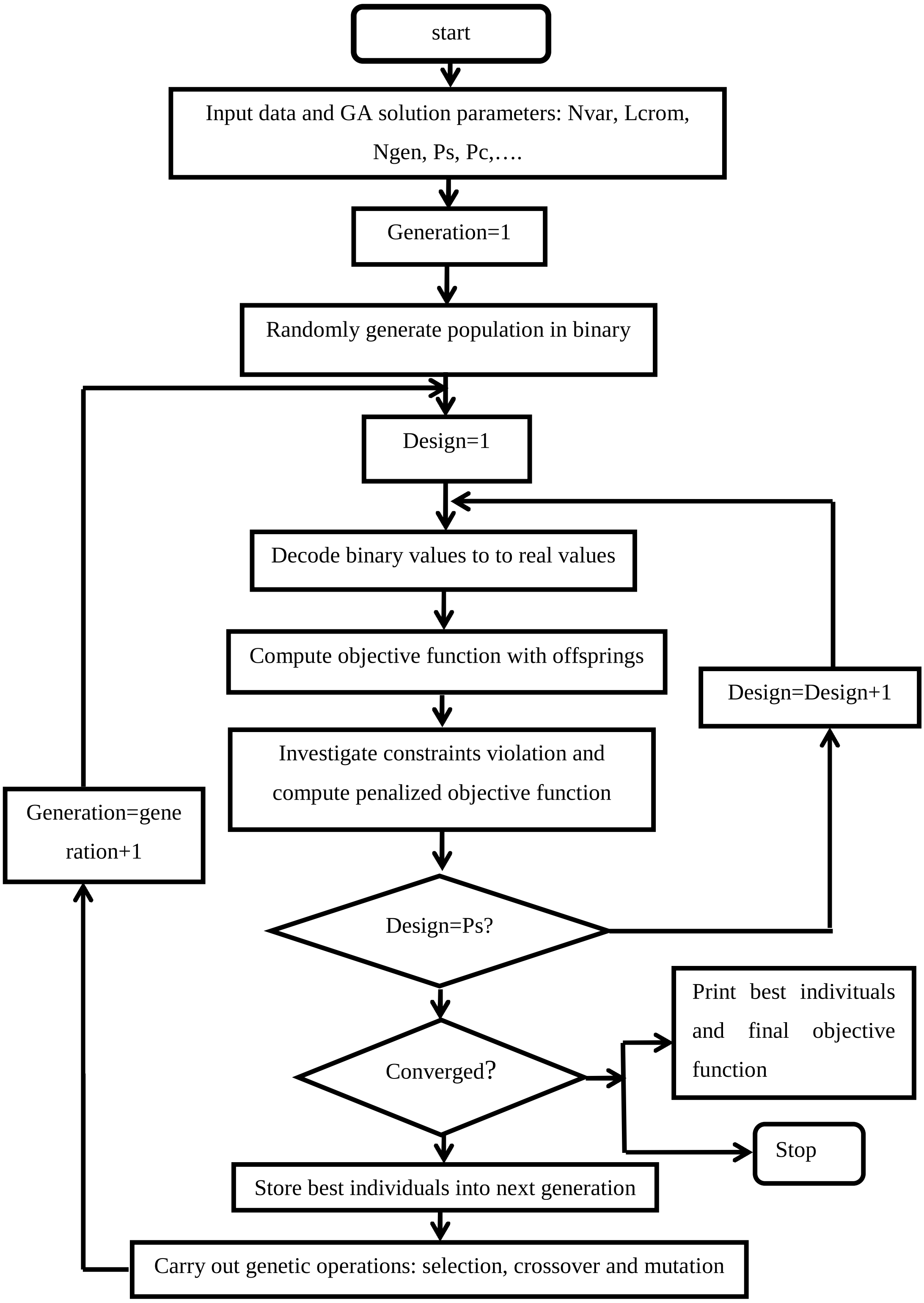


Figure.4.6 Structure chart for simple Genetic Algorithm.

transformation from is used for a constant case of unconstrained problems by applying penalty based transformation methods. Various approaches to applying penalty functions are found in the literature.

The one method of transformation in such away violated constraints is normalized explained by Goldberg, sum of squared, and then a penalty coefficient is multiplied to the summation. This sum is then added to the actual objective function allowing the constrained problem to select an unconstrained form of the structure. A near method was employed by Rajeev and Krishnamoorthy in 1992 [25]. The method applied by Ghasemi [20] is a little different, as explained in the next sections. To generalize the problem, constraints are stated in normalized form.

Inequality constraints: Dealing with the inequality constraints, the first step is to normalize the constraints by dividing them by the related permissible value of the constraint; thus

$$c_{i,j}=\frac{g_{i,j}}{g_{i,all}} \;, \quad i=1,\ldots n_c \text{ and } j=1,\ldots p_s. \qquad (4.7)$$

$$\bar{g}_{i,j}=c_{i,j}-1.0$$

Where n_c is the number of constraints, p_s is the population size, $g_{i,j}$ is the i^{th} constraint function of j^{th} of population, $g_{i,all}$ is the allowable value of i^{th} constraints and $c_{i,j}$ is the normalized value of the i^{th} constraints for j^{th} population. Therefore the constraints are achieved if $\bar{g}_{i,j} \leq 0$. Here, corresponding to the degree by which the constraint is violated, the modified objective function is penalized. So another parameter ($p_{v,i}$) is taken when the $\bar{g}_{i,j}>0$ then:

$$P_{v,i}=(c_{i,j})^k \qquad (4.8)$$

When k is an integer represents the degree of constraint violation. Here, another parameter $\tilde{g}_{i,j}$ and it is computed from the equation:

$$\tilde{g}_{i,j} = p_c \,.\, p_{v,i} \,.\, (\bar{g}_{i,j})^2 \qquad (4.9)$$

Where p_c is penalty coefficient. Therefore, every constraint has been violated individually with penalized, then additional parameter $\hat{g}_j$ represents the sum is set,where

$$\hat{g}_j = \sum_{i=1}^{n_c} \tilde{g}_{i,j} \tag{4.10}$$

The objective function is not controlled for the value of p_c, it should be matched to the value of the objective function. This problem is solved by adding unity to the sum of $\hat{g}_j$ for all the individual populations in a generation, and also dealing with the current non-penalized objective value and multiplying it by the sum. $\hat{F}_j$ is the penalized objective function can be represented in the equation (4.11) as,

$$\hat{F}_j = F_j . (1+\hat{g}_j) \tag{4.11}$$

Where, the objective function F_j is without any constraint penalization effect.

Equality constraints: An upper and lower limit as the constraint function have, the violation on either side of the limit constraint it is essential that it is treated in the same way. Else, if treatment of one side is penalized more than the other, this makes the population to transport towards the less penalized side. Then, the modification for limits on both sides of the constraint is happening so that balance is reached. The equality constraint violation, where the lower and upper limits closely match, will be handled in exactly the same way, here divergence from a limit to the other part will be penalized equally [20].

4.8 Convergence of Solution

GAs using many convergence criteria in its process. There are three type of convergence criteria was used by Ghasemi et al. [20] in their study, and if only one of them is attained, then the termination of the optimization process is reached. These are as shown below:

- As the percentage difference between the average value of all the designs and the best parent in a population (non-penalized values) achieves a very small defined value of convergence rate. Therefore,

$$\left|\frac{\bar{F}-F_{j(best)}}{\bar{F}}\right| \times 100 \leq \text{convergence rate}$$

Where

$$\bar{F}=\left(\sum_{j=1}^{P_s} F_j\right) \Big/ P_s \tag{4.12}$$

Where $\bar{F}$ is the average fitness value in a generation, $F_{j(best)}$ is the fittest design variable.

- If the fittest design variable not changed for a number of successive generations, or the difference of the fittest design of common generation and that of a number of generations before is a small amount.

- **Maximum generations:** When the required number of generation's has been reached the algorithm is terminated.

- **Elapsed time:** When a required time has reached the termination of the process is obtained. If the maximum number of generations has been reached before the required time has finished, the process will end.

- **Stall generations**: The algorithm stops if there is no improvement in the objective function for a sequence of consecutive generations of length stall generations.

- **Stall time limit**: If there is no improvement in the objective function during the process the algorithm terminates during a period of time in seconds equal to stall time limit.

The criteria of convergence or termination is at the end carries the search to stop. A few methods of termination process are sorted in the following sections.

4.8.1 Best individual

Once the minimum fitness in the population falls below the convergence value when the criteria of best individual convergence is finishing the search. This carries the search to a faster conclusion guaranteeing at least one good solution.

4.8.2 Worst individual

When the least suitable individuals in the population have fitness less than the convergence criteria worst individual terminates the search. This guarantees the entire population to be of minimum set, while the best individual may not be considerably better than the worst. In this case, the strict convergence value may never be happened, in which case the search will terminate after the maximum has been exceeded [52].

4.8.3 Sum of fitness

In this termination method, when the sum of the fittest in the entire population is equal or less than to the value of convergence in the population record, the search is considered to have satisfied the convergency. This guarantees that essentially all individuals in the population will be within a specific fitness interval, while it is better to pair this criteria of convergence with lowest gene replacement, then a few weak individuals in the population will blow out the fitness sum. While setting the convergence value, the population size has to be considered.

4.8.4 Median fitness

Which should give a good range of solutions to choose from, here at least half of the individuals will be better than or equal to the convergence value [52].

4.9 Examples

The theory of GA was explained in the previous sections of this chapter. To perform the previous formulation of the GAs, 3D truss optimization examples are done. The GA is coded with FORTRAN language run by a compiler. Starting the work with a simple frame example and extended it to more complicated examples for transmission tower. This study is done under the static analysis base the FE. The obtained results compared with the results available in the literature solved by other researchers. In all examples of GA for types of design veriables are used. Such as;

- Rectangular cross section (thicknesses are different to the widths of members)
- Square cross section (thicknesses are equal to the widths of members)
- Rectangular cross section (widths of the section are constant and thickness are design variables)
- Rectangular cross section (thicknesses of section are constant and widths are design variables)

4.9.1 Three dimensional four bar truss

Problem definition: The four bar truss of Figure 4.7 is to be optimized for minimum weight. Two design variables are considered by the GA, DV_1 is $A^{(1)}=A^{(2)}$ and DV_2 is $A^{(3)}=A^{(4)}$, where the design variables is the cross-sectional dimensions. The constraints are maximum tensile stress $\sigma_t = 4500.0\ \text{kN/m}^2$, maximum compressive stress $\sigma_c = -2500.0\ \text{kN/m}^2$ and maximum displacement $0.1\ \text{m}$. Material properties for the truss are: Young's modulus $E = 0.2\times10^9\ \text{kN/m}^2$ and material density $\rho = 1.0\ \text{kg/m}^3$. Two horizontal loads of 2.0 KN and $-4.0\ \text{kN}$ are imposed in the x and y-directions at node 5, along with a 3.0 kN upward vertical load [48-49].

The GA pseudo-continuous design variables considered are in the range 0.003 to 0.035 m, population size 100, number of iterations 100, design variable binary string length $m = 8$.

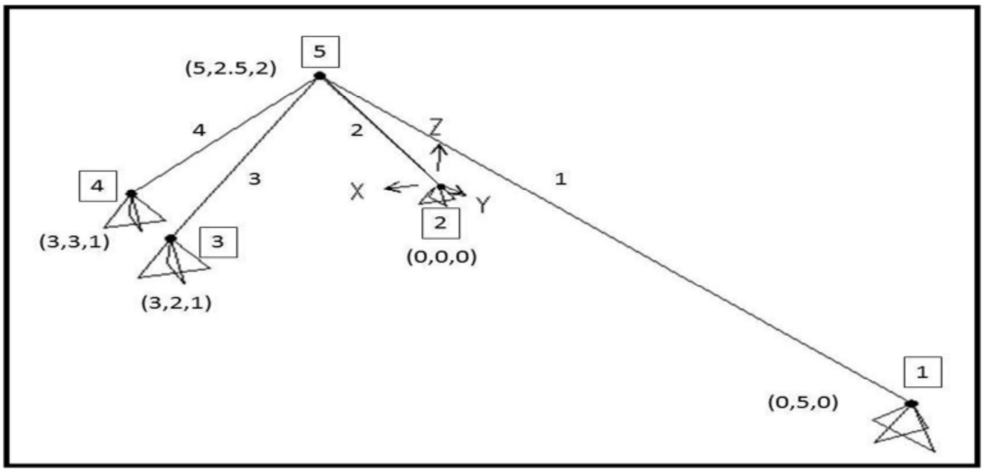


Figure 4.7 Nodal and element numbering for four-bar 3D truss.

Discussion of results: The optimum results are summarized and compared with other solutions, which are available in the literature and shown in Table 4.1. Size optimization resulted in a 13.8 % reduction in the total weight from the initial value of 10.5 kg. The solution for four types of cross section is done. All solution is converged only the fix width not converged (give the best solution) with the convergency of (0.15%). The best optimal design for four-bar truss was achieved after finishing maximum number of generations 3rd case of this solution, the obtained results of the final objective function shown in Table 4.1 below.

Table 4.1 Comparison of optimum static four bar 3D truss against other solutions.

Members	Optimum design variables Area (m^2)								
	Present					Tayşi [47]	Langley [47]	Al-Khamis [48]
	Rectangular Section (4 DV)		Square Section (2 DV)	fixed width (2 DV)	fixed thickness (2 DV)			
	width	thick						
1,2	0.034	0.035	0.00108	0.00117	0.00121	0.001055	0.001102	0.001100
3,4	0.0078	0.034	0.00033	0.00008	0.000574	0.00057	0.000279	0.000290
Opt.W (kg)	9.82		9.15	9.053	10.184	9.677	9.158	9.150

4.9.2 Twenty five bar truss

Problem definition: The twenty five bar 3D truss of Figure 4.8 is to be optimized for minimum weight. The following material properties are used: elastic modulus $E = 1.0 \times 10^4$ ksi and material density, $\rho = 0.1$ lb/in^3. Nodes 7, 8, 9 and 10 are fully constrained, and nodes 1, 2, 3 and 6 are loaded with different load magnitude see Table 4.2. The design constraints are maximum tensile stress $\sigma_t = 40.0$ ksi, maximum compressive stress $\sigma_c = -40.0$ ksi and maximum displacement 0.35 in which is imposed for all nodes and in all directions. Eight groups of members are used are considered by the GA. The member groupings for design variable assignment are shown in Table 4.3 [47-48].

The GA pseudo-continuous design variables considered are in the range 0.316 to 2.236 in, population size 100, number of generation is 100, design variable binary string length $m = 8$.

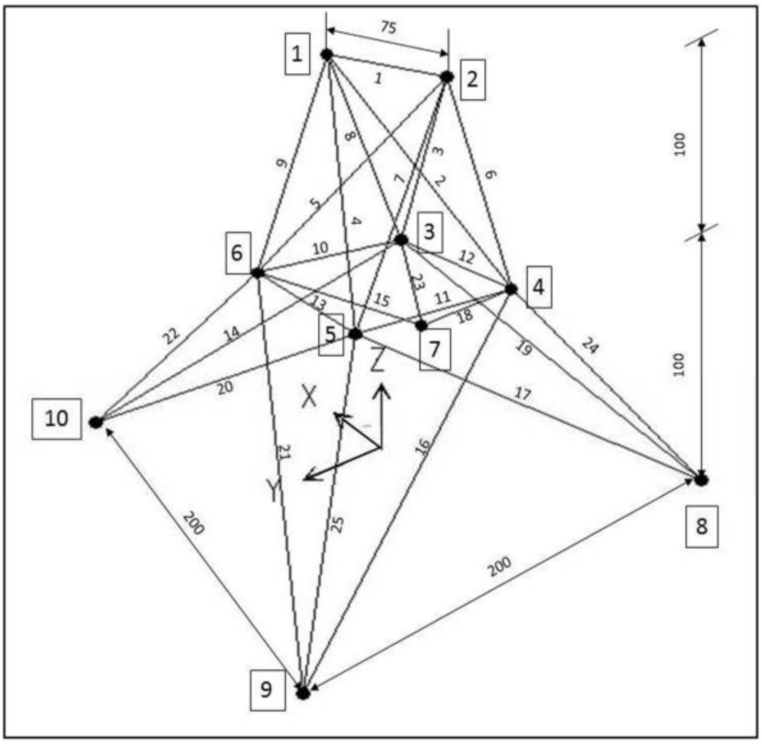


Figure 4.8 Nodal and element numbering for twenty five bar 3D truss.

Table 4.2 Loading details for twenty five bar 3D truss.

Joint	P_x (lb)	P_y (lb)	P_z (lb)
1	1000	-10000	-10000
2	0	-10000	-10000
3	500	0	0
6	600	0	0

Discussion of Results: The resulting truss design of the GA for pseudo-continuous design variables are presented and compared with various references in Table 4.3. Size optimization results with (52.14 %) reduction (for the rectangular solution) from the initial weight value of (992.16 lb). Initial cross sectional areas use in the optimization process is 3.0 in^2. Four section properties is used for the optimization process as tabulated, and different solutions are obtained with the convergency rate of (0.15%). While the algorithm is not converged during all optimization procedure with various cross sections.

Table 4.3 Comparison of optimum static twenty five bar 3D truss against other solutions in literature.

Element number	Present study area (in^2)					Reference solutions area (in^2)		
	Rectangular section DV=16		Square section DV=8	Fixed width DV=8	Fixed thickness DV=8	Tayşi [47]	Langley [47]	Al-Khamis [48]
	Width	Thick.						
1	1.66	1.27	0.29	0.43	2.73	0.1000	0.1000	0.1000
2,3,4,5	1.74	1.96	0.15	4.29	4.15	0.1000	0.2537	0.1000
6,7,8,9	1.17	0.775	0.49	0.58	0.87	3.2600	3.2322	3.5800
10,11	1.66	0.956	0.35	1.94	1.42	0.1000	0.1000	0.1000
12,13	0.549	0.534	1.00	2.75	0.37	2.6300	1.9831	2.0500
14,15,16,17	1.00	0.452	0.285	3.73	0.98	0.8900	0.8686	0.8000
18,19,20,21	0.994	1.68	1.40	3.46	3.19	0.4200	0.2345	0.2200
22,23,24,25	0.768	1.46	1.33	1.27	2.42	3.8900	3.9816	3.8700
Optimum weight (lb)	474.83		243.88	911.54	733.56	488.74	488.74	472.43

4.9.3 3D Transmission Tower

Problem definition: The 3D transmission tower truss of Figure 4.9 with 672 elements and 306 nodes is to be optimized for minimum weight. The following material properties are used: elastic modulus $E = 200 \times 10^9$ N/m^2 and material density, $\rho = 7860$ kg/m^3. Nodes bottom points are fully constrained, and nodes a, b, c and d are loaded with different loads magnitude see Table 4.4. The design constraints are maximum tensile stress $\sigma_t = 400.0 \times 10^6$ N/m^2, maximum compressive stress $\sigma_c = -400.0 \times 10^6$ N/m^2 and maximum displacement $0.1\,\text{m}$ in each node and in all directions.

Table 4.4 Loading for transmission tower.

Node number	Load (N) (Z-direction)
a, b, c	-3850
d	-1900

Initial design cross sectional areas of the members were 0.01 m^2, thickness and width of rectangular cross section of members are considered for the GA pseudo-continuous design variables. Minimum and maximum values of dimensions are 0.05 and 0.15 m respectively. The member groupings for design variable assignment are shown in Figure 4.9 and classified by;

- Rectangular cross section (with 12 thickness and 12 width, totaly 24 design variables)
- Square cross section (width and thickness are equal so totaly 12 design variables)
- Rectangular cross section (width of sections are constant and 12 thickness design variables)
- Rectangular cross section (thickness of sections are constant and 12 width design variables)

The GA pseudo-continuous design variables considered are in the range 0.05 to 0.15 m, population size 100, number of generation is 100, design variable binary string length $m = 8$.

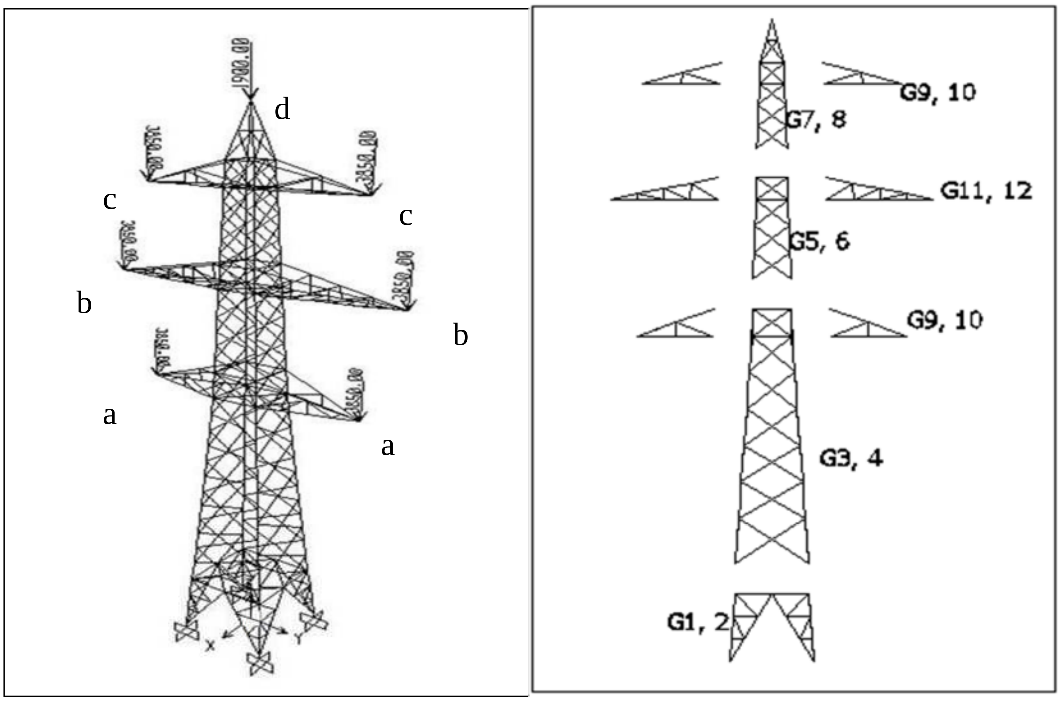


Figure 4.9 Transmission tower loading and grouping of design variables.

Discussion of Results: The resulting tower design of the GA for pseudo-continuous design variables are presented in Table 4.5. The optimum structure weight is obtained for various sections in square meters with the algorithm. For initial design cross sectional areas of the members were 0.01 m^2 and initial total weight is 48606 Kg. Percent improvements are shown in Table 3 and, size optimization results with 64.68 % reduction of total weight of the initial value. Figure 4.10 shows the objective function improvements with respect to iteration numbers of a typical optimization problem for rectangular section.

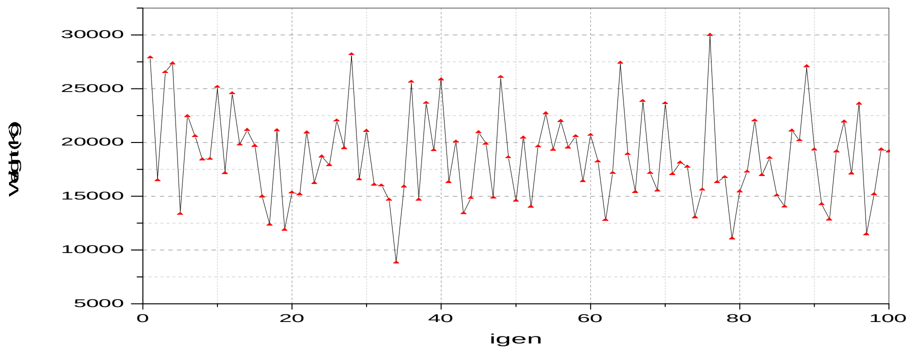


Figure 4.10 Show relation between generations no. and objective function for tower using rectangular sections.

Table 4.5 Comparison of optimum transmission towers with various design variables

Element group	Present GA (m^2)				
	Rectangular section DV=24		Square section DV=12	Width fixed section DV=12	Thickness fixed section DV=12
	Width (m)	Thick. (m)			
G_1	0.0438	0.0123	0.00453	0.0081	0.00751
G_2	0.0943	0.0925	0.0141	0.0119	0.00916
G_3	0.123	0.0966	0.00924	0.00022	0.00721
G_4	0.0348	0.0375	0.00043	0.00697	0.0102
G_5	0.123	0.0619	0.00022	0.0066	0.00539
G_6	0.110	0.0623	0.011	0.00557	0.0116
G_7	0.028	0.019	0.00697	0.00295	0.00344
G_8	0.027	0.067	0.00873	0.0115	0.00152
G_9	0.012	0.108	0.0108	0.00533	0.00022
G_{10}	0.015	0.069	0.00645	0.00447	0.00405
G_{11}	0.026	0.033	0.00127	0.00405	0.00447
G_{12}	0.123	0.01	0.0135	0.0103	0.0115
Optimum weight (Kg)	17163.85		39729.30	33795.14	44085.71
Percent improvements	64.68		18.26	30.47	9.30

4.10 Summary and Conclusions

The GA is coded in FORTRAN language to run the algorithm process for the search optimization. Size optimization is the aim of this study to minimize the weight's of the structure as objective function. The optimum design of the structures is obtained under stress and displacement constraints. The weight of the structures is optimized with significant difference for 3D examples. The weights of four, twenty five and 672 bars are decreased from the initial weight by (13.8%), (52.14%) and (64.7%) respectively for the available solution results. The results obtained from this chapter encourages to expand the study of the future works. The design variables were cross section dimensions, while the solution is done for various types of sections. The algorithm searches all the available solution under the constraints of allowable stress and displacement to converge solution or find the best solution. The best solution is selected among all available results satisfying the constraints by the algorithm.

CHAPTER 5

DESIGN BY COMPUTER AIDED SOFTWARE

5.1 Introduction

The features of today software's are powerful and completely integrated modules for the design of steel and reinforced concrete structures. The program provides the user with options to create, modify, analyse and design structural models, all from within the same user interface. The software programs are capable of performing initial member sizing and optimization from within the same interface.

The software programs provide an interactive environment in which the user can study the stress conditions, make appropriate changes, such as revising member properties, and re-examine the results without the need to re-run the analysis. Detailed design information brings up with a single mouse click on an element. For design purposes members can be grouped together. Both graphical and tabulated formats can be readily printed as output results.

The software programs are structured to support a wide variety of the latest international and national design codes for the automated design and check of concrete and steel frame members. The programs currently support the following steel design codes especially the software used in this study [52],

• U.S. AISC/ASD (1989),
• U.S. AISC/LRFD (1994),
• U.S. AASHTO LRFD (1997)
• Canadian CAN/CSA-S16.1-94 (1994),
• British BS 5950 (1990), and
• Eurocode 3 (ENV 1993-1-1).

5.2 Engineering Design

The design of a structural member entails the selection of a cross section that will safely and economically resist the applied loads. The economy usually means minimum weight that is, the minimum amount of steel. This amount corresponds to the cross section with the smallest weight per foot, which is the one with the smallest cross-sectional area. Although other considerations, such as ease of construction, may ultimately affect the choice of member size, the process begins with the selection of the lightest cross-sectional shape that will do the job. Having established this objective, the designer must decide how doing it safely, which is where different approaches to design come into play. The fundamental requirement of structural design is that the required strength not exceed the available strength, thus the required strength must be less than the available strength to ensure that the structure is within safe situation to resist the applied loads.

For example the Allowable Strength Design (ASD), the member was selected under the consideration of it has cross-sectional properties which give proper area and moment of Inertia that are enough to resist the maximum applied load, torsion and bending moment that not exceeding the allowable. This allowable value is obtained by dividing the nominal strength by a factor of safety. Strength can be an axial force strength (as in tension or compression members), a flexural strength (moment strength), or a shear strength.

If stresses are used instead of forces or moments, the relationship becomes the maximum applied stress must be less than allowable stress. This approach is called allowable stress design. The allowable stress will be in the elastic range of the material [54].

5.3 Design Load Combinations According to AISC-ASD (89)

The design load combinations are used to find different combinations of the various load cases for which the structure needs to be make design and check the structure according to the selected standard by the user. The load combination factors are used and defined in the selected design code, but here in this study the dead loads are used only to make comparisons between the program and the software and I try to in this modelling to be near to that modelled by the program in the previous chapter.

The combinations of design load are the several combinations of the load cases for making the required checking for the structure. When using the AISC-ASD89 code, if a structure is exposed to Dead Load (DL), Live Load (LL), Wind Load (WL), and Earthquake caused Load (EL), and taking into account that wind and earthquake forces are reversible, also the combination of the following load may have to be defined (ASD section A4) [53,54].

$$\begin{gathered} DL \\ DL+LL \\ DL\pm WL \\ DL+LL\pm WL \\ DL\pm EL \\ DL+LL\pm EL \end{gathered} \qquad (5.1)$$

These combinations must be considered in the design of steel structure and take the critical effect of these combinations. In this study the first combination of equation 5.1 is considered and automatic selections for cross sections are performed.

5.4 Classification of Sections According to ASD-89

The allowable stresses for axial compression and flexure are dependent on the classification of sections as compact, noncompact, slender, or too slender. The way of classifying the individual members according to the limiting width/thickness ratios given in sections (ASD B5.1, A-B5-2). The definition of the section properties required is given in Figure 5.1 and Table 5.1 for single angles as shown [53]

Table 5.1 Limiting width-thickness ratios for classification of sections based on AISC-ASD-89.

Section Description	Ratio Checked	Compact Section	Noncompact Section	Slender Section
Angle	b/t	Not Applicable	$\leq 76/\sqrt{F_y}$	No limit

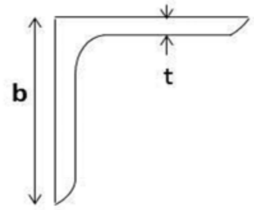


Figure 5.1 Single angle dimensions.

5.5 Stress Calculations with ASD-89 Manual

The distribution of internal force systems across the structural member cross section in the form of stresses. But, there are only two types of stress: first type which works vertically to the member cross section and the second which works laterally. The first is known as a direct stress, the latter as a shear stress. These stresses are distributed over the structural member cross section area depends on the internal force system on the section and also upon the cross section geometry.

The calculated stress for the member in non-slender section is calculated for each load combination based on the gross cross-sectional properties [52].

$$
\begin{aligned}
f_a &= \frac{P}{A} \\
f_{b33} &= \frac{M_{33}}{S_{33}} \\
f_{b22} &= \frac{M_{22}}{S_{22}} \\
f_{v2} &= \frac{V_2}{A_{v2}}
\end{aligned}
\tag{5.2}
$$

$$f_{v3}=\frac{V_3}{A_{v3}}$$

If the section is slender with slender stiffened elements, like slender web in I, channel, and box sections or slender flanges in a box, effective section moduli based on reduced web and reduced flange dimensions are used in calculating stresses(ASD A-B5.2d) [53].

For Single-angle sections, the design considers the principal properties and the shear stresses are calculated for directions along the geometric axes.

5.5.1 Allowable stresses calculations

The allowable stresses in compression, tension, bending, and shear are computed for compact, noncompact, and slender sections according to ASD-89 design manual. For the angle sections, the principal axes are determined and all computations related to flexural stresses are based on that. The allowable stress can be explained according to member internal forces to the following [53];

5.5.1.1 Allowable stress in tension

The allowable axial tensile stress value F_a are assumed to be

$$F_a = 0.6F_y \tag{5.3}$$

For members in tension, which have *l/r* greater than 300, a message to that effect is printed (ASD B7). For single angles, the minimum radius of gyration, r_z , is used instead of r_{22} and r_{33} to compute *l/r* [53,55].

5.5.1.2 Allowable stress in compression

The allowable axial compressive stress is the minimum value obtained from flexural buckling and flexural-torsional buckling. The allowable compressive stresses are determined according to the following description. For members in compression, which have *Kl/r* is greater than 200, a warning message is printed (ASD B7). For single angles, the minimum radius of gyration, r_z , is used instead of r_{22} and r_{33} to compute *Kl/r* [52,55].

5.5.1.3 Flexural buckling

The allowable axial compressive stress value, F_a , depends on the slenderness ratio *Kl/r* based on gross section properties and a corresponding critical value, C_c , where (ASD E2) [53],

$$\frac{Kl}{r} = \max\left\{\frac{K_{33}l_{33}}{r_{33}}, \frac{K_{22}l_{22}}{r_{22}}\right\},$$

$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}} \tag{5.4}$$

For single angles, the minimum radius of gyration, r_z is used instead of r_{22} and r_{33} in computing K*l/r*. For compact, or noncompact section F_a are calculated as follows.

When $Kl/r \leq C_c$ [53]

$$F_a = \frac{\left\{1.0 - \frac{\left(Kl/r\right)^2}{2C_c^2}\right\} F_y}{\frac{5}{3} - \frac{3\left(Kl/r\right)}{8C_c} - \frac{\left(Kl/r\right)^3}{8C_c^3}} \tag{5.5}$$

Or $Kl/r > C_c$

$$F_a = \frac{12\pi^2 E}{23\left(Kl/r\right)^2} \tag{5.6}$$

If *Kl/r* is greater than 200, then the calculated value of the Fa is taken not to exceed the value of Fa calculated by using the equation ASD E2-2 for compact and noncompact sections (ASD E1). If we have slender sections except slender pipe sections F_a is evaluated as evaluated in the ASD-89 manual for steel design as shown below, when $Kl/r \le C_c'$: (ASD A-B5-11) [53].

$$F_a = Q\frac{\left\{1.0 - \frac{\left(Kl/r\right)^2}{2C_c^2}\right\}F_y}{\frac{5}{3} - \frac{3\left(Kl/r\right)}{8C_c} - \frac{\left(Kl/r\right)^3}{8C_c^3}} \tag{5.7}$$

If $Kl/r > C_c'$, F_a is evaluated by using the equation according to ASD-89 (ASD A-B5-12)

$$F_a = \frac{12\pi^2 E}{23\left(Kl/r\right)^2} \tag{5.8}$$

For calculation in this case we use this equation to calculate the corresponding critical value as evaluated in the section ASD (A-B5.2c)

$$C_c' = \sqrt{\frac{2\pi^2 E}{QF_y}} \tag{5.9}$$

For slender sections, if *Kl/r* is greater than 200, then the calculated value of the Fa is taken not to exceed its value calculated by using the equation ASD A-B5-12 (ASD B7, E1).

5.5.1.4 Flexural-torsional buckling

The allowable axial compressive stress value Fa is determined by the limit states of torsional and flexural-torsional buckling is determined according to section (ASD E3, C-E3) of ASD-89 for $(Kl/r)_e \leq C_c'$.

$$F_a = Q\frac{\left\{1.0 - \frac{(Kl/r)_e^2}{2C_c'^2}\right\}F_y}{\frac{5}{3} - \frac{3(Kl/r)_e}{C_c'^2} - \frac{(Kl/r)_e^3}{8C_c'^3}} \tag{5.10}$$

If the $(Kl/r)_e > C_c'$ the F_a is calculated as evaluated in ASD-89 section (E2-2, A-B5-12) as shown in equation below [52, 53].

$$C_c' = \sqrt{\frac{2\pi^2 E}{QF_y}} \tag{5.11}$$

While

$$(Kl/r)_e = \sqrt{\frac{\pi^2 E}{F_e}} \tag{5.12}$$

Where F_e can be calculated as mentioned in ASD commentary section (ASD C-E3) referring to the 1986 version of the AISC-LRFD code for this F_e is calculated with the software SAP2000 for the single angle section with equal legs as follows.

$$F_e = \left(\frac{F_{e33}+F_{ez}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{e33}F_{ez}H}{(F_{e33}+F_{ez})^2}}\right] \tag{5.13}$$

When we have single angle sections with unequal legs, F_e is calculated as the minimum real root of solving the following cubic equation according to LRFD section (A-E3-7).

$$(F_e\text{-}F_{e33})(F_e\text{-}F_{e22})(F_e\text{-}F_{ez})\text{-}F_e^2(F_e\text{-}F_{e22})\frac{x_o^2}{r_o^2}=0 \qquad (5.14)$$

Where x_o, y_o are the coordinates of the shear centre with respect to the centroid of the cross sectional area, and r_o is the polar radius of gyration about the shear centre of the cross section can be represented by the equation below.

$$r_o=\sqrt{x_o^2+y_o^2+\frac{I_{22}+I_{33}}{A_g}} \qquad (5.15)$$

$$H=1\text{-}\left(\frac{x_o^2+y_o^2}{r_o^2}\right) \qquad (5.16)$$

$$F_{e33}=\frac{\pi^2E}{\left(K_{33}l_{33}/r_{33}\right)^2} \qquad (5.17)$$

$$F_{e22}=\frac{\pi^2E}{\left(K_{22}l_{22}/r_{22}\right)^2} \qquad (5.18)$$

$$F_{ez}=\left[\frac{\pi^2EC_w}{(K_zI_z)^2}+GJ\right]\frac{1}{Ar_o^2} \qquad (5.19)$$

K_{22}, K_{33} are effective length factors in minor and major directions, K_z is the effective length factor for torsional buckling, and it is taken equal to K_{22} in the software SAP2000, l_{22}, l_{33} are effective lengths in the minor and major directions, l_z is the effective length for torsional buckling, and it is taken equal to l_{22} .While for angle sections, the principal moment of inertia and radii of gyration are used for computing F_e according to (ASD SAM 4). Also, the maximum value of *Kl*, i.e, max(K_{22} l_{22} ,K_{33} l_{33}), is used in place of K_{22} l_{22} or K_{33} l_{33} in calculating F_{e22} and F_{e33} for the angle section case [52].

5.5.1.5 Allowable stress in bending

There are many criteria's that affect the allowable stress in bending for a member of the structure such as the geometric shape of the cross-section, the axis of bending, the compactness of the section and a length parameter. The allowable flexural stresses for single angle are calculated based on their principal axes of bending according to (ASD SAM5.3).

For the major axis of bending the allowable stress is the minimum considering the limit state of lateral-torsional buckling and local buckling according to (ASD SAM 5.1). The allowable major bending stress for single-angles for the limit state of lateral torsional buckling is given according to (ASD SAM 5.1.3) as follows for the limitation of $F_{ob} \leq F_y$.

$$F_{b,major}=\left[0.55-0.10\frac{F_{ob}}{F_y}\right]F_{ob} \tag{5.20}$$

When the elastic lateral-torsional buckling stress (F_{ob}) is exceeds the yield stress (F_y) the equation (5.21) is applied instead of (5.20)

$$F_{b,major}=\left[0.95-0.50\frac{F_y}{F_{ob}}\right]F_y \leq 0.66F_y \tag{5.21}$$

Where, F_{ob} is the elastic lateral-torsional buckling stress as calculated below. The elastic lateral torsional buckling stress, F_{ob} , for equal-leg angles is taken as

$$F_{ob}=C_b\frac{28,250}{l/t} \tag{5.22}$$

In the case of using angle sections with unequal leg's section the F_{ob} is calculated by the equation.

$$F_{ob}=143{,}100C_b \frac{I_{min}}{S_{major}l^2}\left[\sqrt{\beta_w^2+0.052\left(\frac{lt}{r_{min}}\right)^2}+\beta_w\right] \quad (5.23)$$

Where l is max (l_{22} , l_{33}), I_{min} is minor principal moment of inertia, I_{max} is major principal moment of inertia, S_{major} is major section modulus for compression at the tip of one leg, r_{min} is radius of gyration for minor principal axis, and

$$\beta_w=\left[\frac{1}{I_{max}}\int_A z\left(w^2+Z^2\right)dA\right]-2z_o \quad (5.24)$$

z is coordinate along the major principal axis, w is coordinates along the minor principal axis, and z_0 is coordinate of the shear centre along the major principal axis with respect to the centroid. β_w is a special section property for angles. It is positive for short leg in compression, negative for long leg in compression, and zero for equal-leg angles (ASD SAM 5.3.2). However, for conservative design of SAP2000 software, it is always taken as negative for unequal-leg angles. In the above expressions C_b is calculated by the equation below;

$$C_b=1.75+1.05\left(\frac{M_a}{M_b}\right)+0.3\left(\frac{M_a}{M_b}\right)^2 \leq 1.5 \quad (5.25)$$

M_a and M_b are the end moments of any unbraced segment of the member and M_a is numerically less than M_b ; M_a/M_b being positive for double curvature bending and negative for single curvature bending. Also, if any moment within the segment is greater than M_b ,C_b is taken as 1.0. Also, C_b is taken as 1.0 for cantilevers and frames braced against joint translation (ASD F1.3). SAP2000 defaults C_b to 1.0 if the unbraced length, l_{22} of the member is redefined by the user (i.e. it is not equal to the length of the member). The user can overwrite the value of C_b for any member by specifying it [52, 53].

In the case of bending, the allowable stress calculated for single angles for the limit state of local buckling according to ASD SAM section (5.1.1) with the limitation for $b/t \leq 65/\sqrt{F_y}$ as shown below.

$$F_{b,major} = 0.66F_y \quad (5.26)$$

If the ratio is between the limitation $65/\sqrt{F_y} < b/t \leq 76/\sqrt{F_y}$

$$F_{b,major} = 0.60F_y \quad (5.27)$$

Otherwise

$$F_{b,major} = Q\,(0.60F_y) \quad (5.28)$$

Where

t = thickness of the leg under consideration,

b = length of the leg under consideration, and

Q = slenderness reduction factor for local buckling.

For the minor axis of bending The allowable stress for single angle is given according to ASD SAM (5.1.1,5.3.1b,5.3.2b) with respect to comparing the ratio and limitations in the case when $b / t \leq 65 / \sqrt{F_y}$.

$$F_{b,minor} = 0.66F_y \quad (5.29)$$

If the ratio is between the limitation $65/\sqrt{F_y} < b/t \leq 76/\sqrt{F_y}$

$$F_{b,minor} = 0.60F_y \quad (5.30)$$

Otherwise

$$F_{b,minor} = Q\,(0.60F_y) \quad (5.31)$$

5.5.1.6 Allowable shear stress

The shear stress is calculated along the geometric axes for the section. For Single-angle sections, principal axes do not coincide with the geometric axes. The allowable shear stress for the single angle section at the major and minor axis of bending in SAP2000 is taken as [52].

$$F_v=0.4F_y \tag{5.32}$$

5.5.2 Allowable stress ratio's for combined stress

In the calculation of the axial and bending stress capacity ratios, first, for each station along the length of the member, the actual stresses are calculated for each load combination. Then the corresponding allowable stresses are calculated. Then, the capacity ratios are calculated at each station for each member under the influence of each of the design load combinations. The controlling capacity ratio is then obtained, along with the associated station and load combination. A capacity ratio greater than 1.0 indicates an over stress [52].

5.5.2.1 Axial and bending stress

With the computed allowable axial and bending stress values and the factored axial and bending member stresses at each station, an interaction stress ratio is produced for each of the load combinations as follows (ASD H1, H2): [53]

- If f_a is compressive and f_a / F_a > 0.15, the combined stress ration is given by the larger of .

$$\frac{f_a}{F_a}+\frac{C_{m33}f_{b33}}{\left(1-\frac{f_a}{F'_{e33}}\right)F_{b33}}+\frac{C_{m33}f_{b22}}{\left(1-\frac{f_a}{F'_{e33}}\right)F_{b22}} \tag{5.33}$$

$$\frac{f_a}{Q(0.6F_y)}+\frac{f_{b33}}{F_{b33}}+\frac{f_{b22}}{F_{b22}} \tag{5.34}$$

Where f_a is axial stress either in compression or in tension. f_{b33}, f_{b22} are normal stress in major and minor direction of bending. F_a is the allowable axial stress. F_{b33}, F_{b22} are allowable major and minor bending stresses. While C_{m33} and C_{m22} are coefficients which they representing the distribution of moment along the member length.

For sway frame C_m has taken 0.85, When if we have nonsway frame with transverse loads the C_m is taken as ($0.6-0.4M_a / M_b$), Where M_a / M_b is the ratio of the smaller to the larger moment at the ends of the member, However it will be positive for double curvature bending, and negative for single curvature bending. For nonsway frame with transverse load and end restrained compression member C_m is taken 0.85 or is taken 1.0 for nonsway frame with transverse load and end unrestrained compression member. If the unbraced length l, is not equal to the length of the member, the user can overwrite the value of C_m for any member. Assumes C_{m22} and C_{m33} associated with the major and minor directions [53].

- If f_a is compressive and the ratio $f_a / F_a \leq 0.15$, a relatively simplified formula is used for the combined stress ratio.

$$\frac{f_a}{F_a} + \frac{f_{b33}}{F_{b33}} + \frac{f_{b22}}{F_{b22}} \qquad (5.35)$$

$$\frac{f_{b33}}{F_{b33}} + \frac{f_{b22}}{F_{b22}} \qquad (5.36)$$

Where either F_{b33} or F_{b22} must be not less than $0.6F_y$ in the first equation (ASD H2-1). The second equation imagines flexural buckling without any beneficial effect from axial compression. For single angle sections, the combined stress is calculated based on the properties about the principal axis (ASD SAM 5.3,6.1.5). These principal axis are determined in SAP2000 [52].

5.5.3 Shear stresses

From the allowable shear stress values and the factored shear stress values at each station, shear stress ratios for major and minor directions are computed for each of the load combinations as follows.

$$\frac{f_{v2}}{F_v} \qquad (5.37)$$

$$\frac{f_{v3}}{F_v} \qquad (5.38)$$

For single angle sections, the shear stress ratio is calculated for directions along the geometric axis. For all other sections the shear stress is calculated along the principal axes which coincide with the geometric axes [52-53].

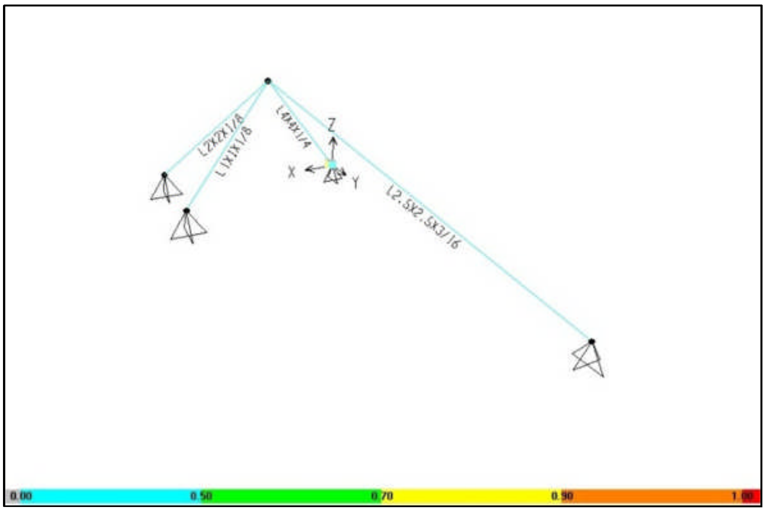
5.6 Examples of SAP2000 Using (AISC-ASD-89) Manual

The GA search optimization technique is explained in chapter four, and the result is discussed about the minimum weight as objective function. The illustration of that chapter is reviewed by SAP2000 software to look to the weight's of the structure in the real point of view. SAP2000 software takes the effect of buckling in checking the structure according to the selected design manual. It performs the optimization by automatic selection of the member sections of the structure. This result gives us significant idea about the buckling criteria in the design of this type of structures, and how much the weight is changed. 3D frames design for optimum with of SAP2000 using (AISC-ASD-89) manual are done in this section.

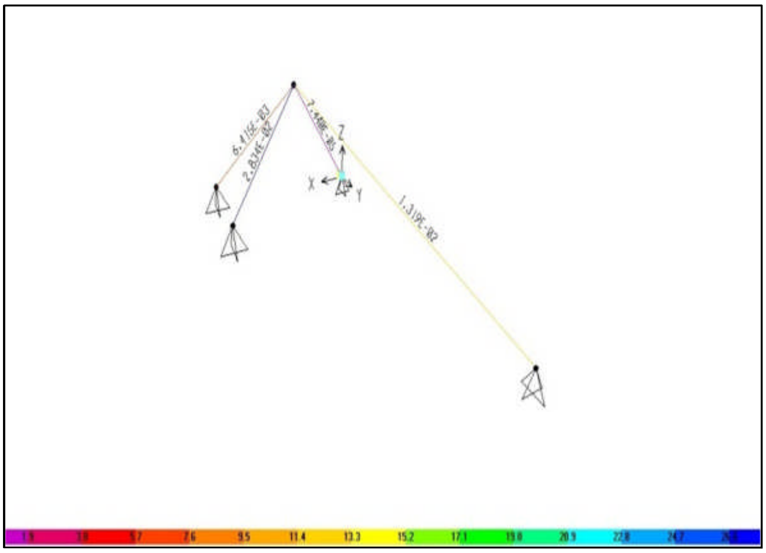
5.6.1 Three dimensions four bar truss

Problem definition: The four bar truss illustrated in section 3.8.1 is to be designed for minimum weight by SAP2000. The maximum tensile and compressive stresses was computed by SAP2000. The maximum displacement $0.1\,\text{m}$ is imposed to joint (5).

The material properties for the truss are: Young's modulus $E = 0.2 \times 10^9\,\text{kN/m}^2$ and material density $\rho = 1.0\,\text{kg/m}^3$. Two horizontal loads of 2.0 and $-4.0\,\text{kN}$ are imposed in the x and y-directions at node 5, along with a 3.0 KN upward vertical load [41,47].



a) Selected sections of the design



b) Virtual work diagram of static load condition.

Figure 5.2 3D four bar frames.

Discussion of Results: When the analysis of the frame is done under static load condition and the group of members cross section is selected. This selection is performed automatically by SAP2000, checking the (AISC-ASD-89) requirements and limitations. The load conditions of point and gravity load is taken in combination to select required sections of the structure. The weight obtained by SAP2000 is

(59.78 Kg) using standard angle sections provided in the software. The structure is checked as shown in Figure 5.2 after a number of iterations and the software give us a message that all steel frames are safe as shown in Figure 5.3 below:

Table 5.2 Summary of sections of design for 3D four bars.

Frame	Design Sections	Design Type	Error Message	Warning Message
ID	in	Text	Text	Text
1	L2.5X2.5X3/16	Brace	No Messages	No Messages
2	L4X4X1/4	Brace	No Messages	No Messages
3	L1X1X1/8	Brace	No Messages	No Messages
4	L2X2X1/8	Brace	No Messages	No Messages

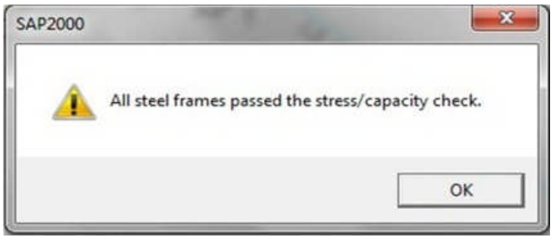


Figure5.3 A message show that all steel frames passed the stress capacity check.

5.6.2 Three dimensions 25 bars frame

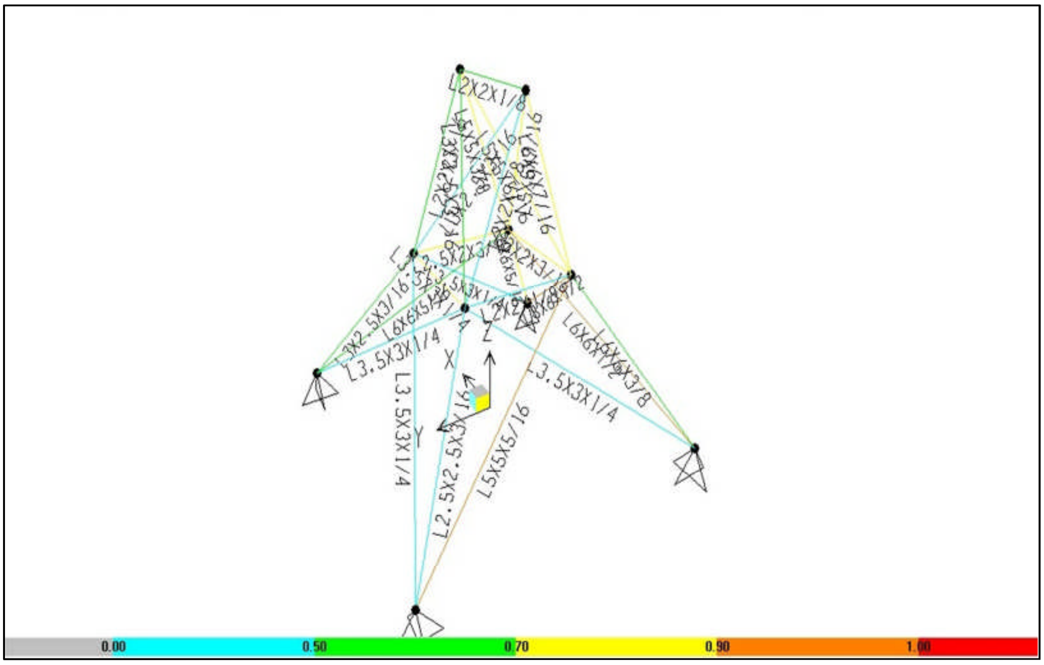
Problem definition: The twenty five bar 3D trusses of section 3.8.2 are to be designed for minimum weight by SAP2000. The following material properties are used: elastic modulus $E = 1.0 \times 10^4$ ksi and material density, $\rho = 0.1$ lb/in^3. Nodes 7, 8, 9 and 10 are fully constrained, and nodes 1, 2, 3 and 6 are loaded with different load magnitudes see Table 5.3. The maximum allowable stress in compression and tension are calculated by the program according (AISC-ASD-89). The maximum displacement of 0.35 in is imposed for all nodes and in all directions. The group of single angles was selected for the members of the structure in design assignment processes.

Table 5.3 Loading details for twenty five bar 3D truss.

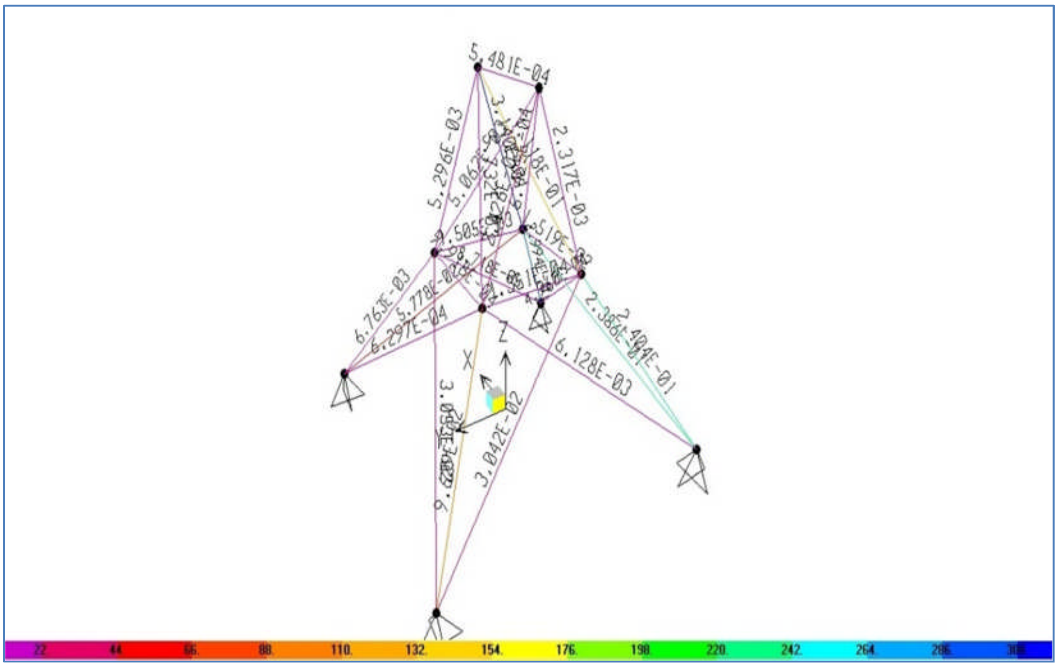
Joint	P_x (lb)	P_y (lb)	P_z (lb)
1	1000	-10000	-10000
2	0	-10000	-10000
3	500	0	0
6	600	0	0

Discussion of Results: The twenty five bar design is illustrated with running this example in SAP2000 after the analysis is done. The member sections is selected automatically by the software for minimum sections according to (AISC-ASD-89) design manual. The entered load combination for point and gravity is described to the software in the analysis and design of the structure. The first trial for the design of the structure is done.

The program checks the structure automatically with entered geometry, material definition and search the group sections selected to each member of a proper section. When the design is complete and the program is selecting the section for each member. When checking the members design details with the SAP2000, there is a warning message for five members that kl/r exceeds 200, while for stress capacity all members passes the check. The second trial is done to change the members with warning error, and this trial will be continued even all members is checked as shown in Figure 5.4. The results of selected sections are tabulated and shown in Table 5.4. The weight of twenty five bars is (0.345 Kip) when taking the effect of buckling in check for members of the structure.



a) Selected sections of the design



b) Virtual work diagram for static load condition.

Figure 5.4 3D twenty five bars frame.

Table 5.4 Summery of 3D twenty five bar steel designs.

Frame	Design	Design Type	Error	Warning
ID	in	Text	Text	Text
1	L2X2X1/8	Beam	No Messages	No Messages
2	L5X5X3/8	Brace	No Messages	No Messages
3	L5X5X7/16	Brace	No Messages	No Messages
4	L3X2X3/16	Beam	No Messages	No Messages
5	L5X5X5/16	Brace	No Messages	No Messages
6	L6X6X7/16	Brace	No Messages	No Messages
7	L2X2X3/16	Brace	No Messages	No Messages
8	L3X2.5X3/1	Brace	No Messages	No Messages
9	L3.5X3X1/4	Beam	No Messages	No Messages
10	L3X2.5X3/1	Brace	No Messages	No Messages
11	L2X2X1/8	Brace	No Messages	No Messages
12	L2.5X2X3/1	Beam	No Messages	No Messages
13	L2X2X1/8	Beam	No Messages	No Messages
14	L2.5X2.5X3/	Brace	No Messages	No Messages
15	L5X5X5/16	Brace	No Messages	No Messages
16	L3.5X3X1/4	Brace	No Messages	No Messages
17	L6X6X5/16	Brace	No Messages	No Messages
18	L3X2.5X3/1	Brace	No Messages	No Messages
19	L3.5X3X1/4	Brace	No Messages	No Messages
20	L6X6X5/16	Brace	No Messages	No Messages
21	L3.5X3X1/4	Brace	No Messages	No Messages
22	L8X6X1/2	Brace	No Messages	No Messages
23	L6X6X1/2	Brace	No Messages	No Messages
24	L6X6X3/8	Brace	No Messages	No Messages
25	L3.5X3X1/4	Brace	No Messages	No Messages

5.6.3 Three dimensions 672 bars transmission tower

Problem definition: The 3D transmission tower illustrated in section 3.8.3 with 672 elements and 306 nodes is to be designed for minimum weight by SAP2000. The following material properties are used: elastic modulus $E = 200 \times 10^9$ N/m^2 and material density, $\rho = 7860$ kg/m^3. Bottom nodes are fully constrained, and nodes a, b, c and d are loaded with different load magnitude see Table 5.5. The design allowable maximum stresses are calculated by the program and maximum displacement $0.1\,\text{m}$ which is imposed for all nodes and in all directions. The member cross section is grouped for automatic selection in design assignment processes.

Table 5.5 Loads of transmission tower.

Node numbers	Load (N) (Z-direction)
a, b, c	-3850
d	-1900

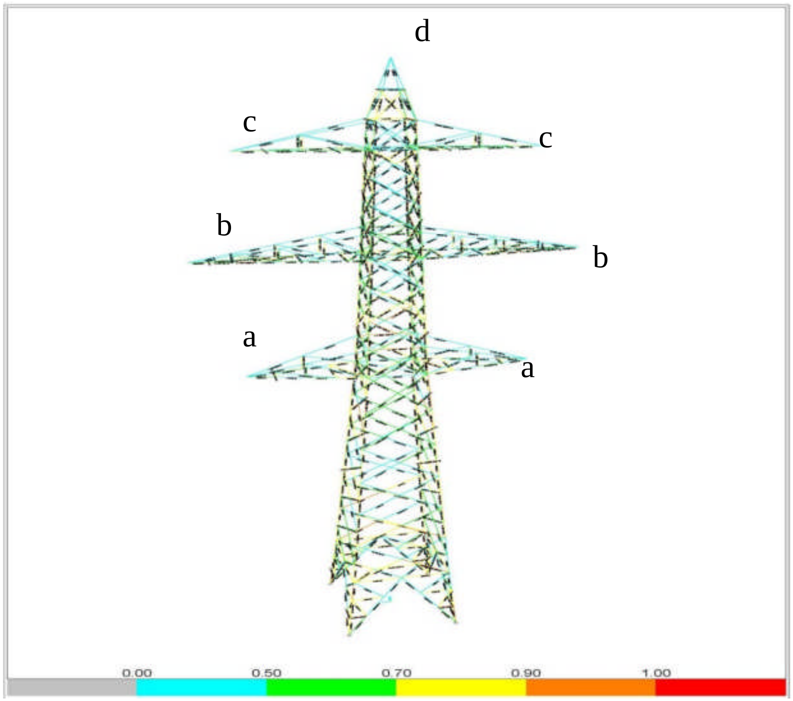


Figure5.5 Selected sections for the 3D 672 bars frame for static load condition.

Discussion of Results: The 672 bar structure is illustrated in this section. The load cases are defined for applying point and gravity loads on the structure. and a group of angle sections is selected. The automatic selection for a cross section of members is performed by SAP2000; beside this members are checked for buckling criteria according to (AISC-ASD-89).

The first trial is not succeeded in select proper sections for the structure, after a number of trials, all members pass the stress capacity check with single angle sections. The detail of design doesn't give us any warning message about buckling problems with single angle sections. The SAP2000 results are obtained after finishing the design process as shown in Figure 5.5. It gives the weight of (18350.62 Kg) for structural designed sections. Figure 5.6 shows a sample of design members near the foundation and crossarms:

```
AISC-ASD89 STEEL SECTION CHECK
Combo  : gravity+point
Units  : N, m, C



Frame  : 493            Design Sect: L6X6X5/8
X Mid  : 0.509          Design Type: Brace
Y Mid  : 1.101          Frame Type : Moment Resisting Frame
Z Mid  : 17.793         Sect Class : Non-Compact
Length : 1.074          Major Axis : 0.000 degrees counterclockwise from local 3
Loc    : 1.074          RLLF       : 1.000

Area   : 0.004          SMajor : 6.325E-05     rMajor : 0.039          AVMajor: 0.002
IMajor : 5.661E-06      SMinor : 6.325E-05     rMinor : 0.039          AVMinor: 0.002
IMinor : 5.661E-06      ZMajor : 1.139E-04     rMax   : 0.049          E      : 1.999E+11
Ixy    : -3.320E-06     ZMinor : 1.139E-04     rMin   : 0.025          Fy     : 344737894.5
Theta  : 45.000

STRESS CHECK FORCES & MOMENTS
    Location            P          M33         M22          V2          V3           T
    1.074       19016.622     -104.333      17.874     144.330     -14.346       0.064

PMM DEMAND/CAPACITY RATIO
    Governing       Total          P      MMajor      MMinor       Ratio      Status
    Equation        Ratio      Ratio       Ratio       Ratio       Limit       Check
    (H2-1)          0.036  =   0.024  +    0.003  +    0.009       0.950          OK

AXIAL FORCE DESIGN
                        P          fa          Fa          Ft
                    Force      Stress   Allowable   Allowable
    Axial       19016.622 5030003.847  34561703.4 206842736.7

MOMENT DESIGN
                        M          fb          Fb          Fe      Cm       K       L      Cb
                   Moment      Stress   Allowable   Allowable  Factor  Factor  Factor  Factor
    Major Moment  -61.136  611366.061 208221787.0  1337062107   1.000   1.000   1.000   1.000
    Minor Moment   86.413 1960930.863 227527010.4  34561703.4   1.000   1.000   4.000

SHEAR DESIGN
                        V          fv          Fv      Stress      Status           T
                    Force      Stress   Allowable       Ratio       Check     Torsion
    Major Shear   144.330   71587.950 137895157.8       0.001          OK       0.000
    Minor Shear    14.346    7115.510 137895157.8   5.160E-05          OK       0.000
```

a) Summary of design for element no. 493 crossarm

```
AISC-ASD89 STEEL SECTION CHECK
Combo  : gravity+point
Units  : N, m, C



Frame  : 661               Design Sect: L6X6X5/8
X Mid  : 1.538             Design Type: Column
Y Mid  : -1.538            Frame Type : Moment Resisting Frame
Z Mid  : 1.435             Sect Class : Non-Compact
Length : 0.963             Major Axis : 0.000 degrees counterclockwise from local 3
Loc    : 0.963             RLLF       : 1.000

Area   : 0.004             SMajor : 6.325E-05      rMajor : 0.039         AVMajor: 0.002
IMajor : 5.661E-06         SMinor : 6.325E-05      rMinor : 0.039         AVMinor: 0.002
IMinor : 5.661E-06         ZMajor : 1.139E-04      rMax   : 0.049         E      : 1.999E+11
Ixy    : -3.320E-06        ZMinor : 1.139E-04      rMin   : 0.025         Fy     : 344737894.5
Theta  : 45.000

STRESS CHECK FORCES & MOMENTS
    Location            P          M33          M22          V2          V3          T
    0.963       -34662.867      477.386        3.185    -580.676      10.172     -0.059

PMM DEMAND/CAPACITY RATIO
    Governing       Total           P       MMajor       MMinor       Ratio      Status
    Equation        Ratio       Ratio        Ratio        Ratio       Limit       Check
    (H1-3)          0.105  =    0.057  +     0.015  +     0.033       0.950          OK

AXIAL FORCE DESIGN
                        P           fa           Fa           Ft
                    Force       Stress    Allowable    Allowable
    Axial      -34662.867  9168523.831  161449228.5  206842736.7

MOMENT DESIGN
                        M           fb           Fb           Fe       Cm       K       L      Cb
                   Moment       Stress    Allowable    Allowable   Factor  Factor  Factor  Factor
    Major Moment  339.822  3398199.622  227527010.4   798625495.   1.000   1.443   1.000   1.000
    Minor Moment -335.318  7609200.700  227527010.4   330298583.   1.000   1.439   1.000

SHEAR DESIGN
                        V           fv           Fv       Stress      Status           T
                    Force       Stress    Allowable        Ratio       Check     Torsion
    Major Shear   580.676   288015.679  137895157.8        0.002          OK       0.000
    Minor Shear    10.172     5045.340  137895157.8    3.659E-05          OK       0.000
```

b) Summery of design of member (661) leg element near foundation.

Figure 5.6 Summary of 3D transmission tower steel design.

5.7 Summary and conclusions

The features of software for obtaining results quick and accurate at the same time provide the user saving time. Understanding the software bases for analysis and design is the important point to model the real structure. The modelling reality for geometric, load combinations and material physical properties is affecting the design results. The illustration of design examples is done according to given material properties and static load condition for gravity and point imposed load. The design results are obtained with the check of (AISC-ASD-89) design manual. The effect of buckling is checked by SAP2000 and the results are considering these criteria.

CHAPTER 6

CONCLUSIONS AND FURTHER WORK

6.1 Conclusions

The size optimization of transmission tower plays an important role in minimizing the amount of material used in the construction of the structure for economic point of view. If reducing the amount of steel used for one tower is affected on the total amount of steel material which is used in the projects. Optimization algorithm is starts following the implementation of the analysis of the structure. A FORTRAN program which uses the FEMs based numerical analysis was modified. 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 present study is performed for optimization of 3D transmission towers with computational tools have been used for analysis and size optimization of frame structures. The used program was written in University college of Swansea and modified by researchers at the University of Gaziantep [48-49]. A number of examples are illustrated to testing and showing the efficiency of the coded program. Built on these concepts the general conclusion is explained.

The algorithm searches all the available solution under the constraints of allowable stress and displacement to converge solution or find the best solution. The best solution is selected among all available results satisfying the constraints by the algorithm. The design variables were cross section dimensions, while the solution is done for various types of sections. For all design variables significant decrease in weight of material with respect to the stress and displacement constraints were get.

When comparing the result with the literature it’s found that the optimum objective function is close to that obtained by one’s in literature.

6.1.1 Structural analysis

The FE stiffness matrix method is working with accurate results in analysis with three node two Gause Quadrature. In the program for the analysis the FE and numerical method applications are used to find the results. The results obtained from the program are compared with other sources to prove the ability and accuracy. This comparison helps to improve the written program to give better solutions.

6.1.2 Structural optimization and design

The size optimization based on GA to achieve the optimum design of the space frames as a complex structure in a 3D system is performed. The constraints are guiding the GAs to the optimum solution and restrict the search space with feasible domain of available results. The objective functions of the cross section dimension give more flexibility to run the GAs with various types of sections. The GAs uses a strategy of a directed search through a problem space from a variety of points in that space. The evaluation of individuals within a population can be conducted simultaneously, as in nature. It successively evaluates and regenerates the new populations as a trial solution from old populations. Reductions of (13.8%, 52.14% and 64.7%) for the three illustrated examples respectively give great encouragement to optimize structures. These reductions are important to save extra materials in construction projects of transmission towers, consequently serving the economical point.

The features of software in the analysis and design of special and complex structures with application of FEM is wide used. Using this application without a background in analysis and design decreases the degree of accuracy in modeling and obtaining correct results. The source program is used to perform the three illustrations solved with a GA to comparing the results. This program checks the structure members for buckling with (AISC-ASD-89) manual requirements.

6.2 Future Work

Future work is proposed to expand and develop of this study;

1. Increase the dimension of the program to analyze more than (1000) members with three noded two Gause-quadrature. To be able in analysis of large structures of transmission towers.

2. Expanding the FORTRAN code in such a way that covers members buckling check. This concluded from comparing the results of the source program and GA.

3. Including discrete optimization approach in a GA to be able to assign standard used cross sections in the optimization process.

4. The disadvantage of executing optimization with GA is time consuming. This delaying solved by use high performance computing especially for large structures like transmission tower [56].

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