OPTIMUM DESIGN OF DOUBLE-LAYER GRID SYSTEMS: COMPARISON WITH CURRENT DESIGN PRACTICE USING REAL-LIFE INDUSTRIAL APPLICATIONS

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ABSTRACT

OPTIMUM DESIGN OF DOUBLE-LAYER GRID SYSTEMS: COMPARISON WITH CURRENT DESIGN PRACTICE USING REAL-LIFE INDUSTRIAL APPLICATIONS

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Double-layer grid systems are three-dimensional pin-jointed structures, which are generally used for covering roofs having large spans. In this study, evolution strategies method is used to optimize space trusses. Evolution strategies method is a type of evolutionary algorithms, which simulate biological evolution and natural selection phenomenon to find the best solution for an optimization problem. In this method, an initial population is formed by various solutions of design problem. Then this initial population starts to evolve by using recombination, mutation, and selection operators, which are adopted for optimization of space trusses by modifying some parameters. Optimization routine continues for a certain number of generations, and best design obtained in this process is accepted as optimum solution.

OFES, a design and optimization software developed for optimum design of steel frames, is modified in this study to handle space truss systems. By using this software, six design examples taken from real-life industrial applications with element numbers changing between 792 and 4412 are studied. The structural systems defined in examples are optimized for minimum weight in accordance with design provisions imposed by Turkish Specification, TS648. The optimization is performed based on selecting member sizes and/or determining the elevation of the structure and/or setting the support conditions of the system. The results obtained are compared with those of FrameCAD, a software which is predominantly used for design of such systems in national current design practice.

Keywords: Structural Optimization, Space Trusses, Double Layer Grid Systems, Evolution Strategies

ÖZ

ÇİFT KATMANLI UZAY KAFES SİSTEMLERİNİN OPTIMUM TASARIMI: GERÇEK ENDÜSTRİYEL UYGULAMALARI KULLANARAK MEVCUT TASARIM ÇALIŞMALARI İLE KARŞILAŞTIRMA

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Uzay kafes sistemler genellikle geniş açıklıklı çatıları kaplamakta kullanılan üç boyutlu mafsal bağlantılı yapılardır. Bu çalışmada uzay kafes sistemlerinin optimizasyonu için evrim stratejileri yöntemi kullanılmıştır. Evrim stratejileri metodu en iyi tasarımı bulmak için biyolojik evrimi ve doğal seleksiyon kavramını taklit eden evrimsel algoritmaların bir çeşitidir. Bu metotta çeşitli tasarımlardan bir başlangıç popülasyonu oluşturulur. Bu başlangıç popülasyonu daha sonra uzay kafes sistemlerinin optimizasyonu için bazı parametreleri yeniden düzenlenen rekombinasyon, mutasyon ve seçim operatörleri kullanılarak evrilmeye başlanır. Optimizasyon süreci belli bir jenerasyon sayısı oluşana kadar devam eder ve bu süreçte elde edilen en iyi tasarım optimum tasarım olarak kabul edilir.

Önceden çelik çerçevelerin optimum tasarımı için geliştirilen ve bir tasarım ve optimizasyon yazılımı olan OFES, bu çalışmada uzay kafes sistemler için yeniden düzenlendi. Bu yazılım kullanılarak, eleman sayısı 792 ve 4412 arasında değişen, gerçek endüstriyel uygulamalardan alınan altı tasarım örneği denendi. Örneklerde tanımlanan yapısal sistemler Türk Standardı, TS648'de verilen tasarım kriterlerine gore minimum ağırlık için optimize edildi. Optimizasyon sistemin eleman kesitlerinin seçimi ve/veya yapı yüksekliğinin belirlenmesi ve/veya mesnet koşullarının değiştirilmesi baz alınarak uygulandı. Elde edilen sonuçlar, bu tip sistemlerin mevcut ulusal tasarım uygulamalarında sıklıkla kullanılan FrameCAD programıyla elde edilenlerle karşılaştırıldı.

Anahtar Kelimeler: Yapı Optimizasyonu, Uzay Kafes Sistemleri, Evrim Stratejileri

To all causes of this effect

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a	Length of module in x-axis
A	Cross-sectional area of member
A(T)	Spectral acceleration coefficient
A_g	Gross sectional area of member
A_i	Sectional area of i-th member
A_o	Effective earth acceleration coefficient
b	Length of module in y-axis
c	continuous design variable
c’	mutated continuous design variable
d	diameter of bolt
d	discrete design variable
d’	mutated discrete design variable
d_c	diameter of sphere
d_h	diameter of pin hole
d_i	inner diameter of nut
d_{max}	maximum bolt diameter connecting to node
d_o	outer diameter of nut
E	Modulus of elasticity for steel
F(x)	Fitness function
F_{all}	Allowable compression stress of nut
F_{c-all}	Allowable compressive stress according to TS648
F_{t-all}	Allowable tensile stress according to TS648
F_{ult}	Ultimate tensile strength of bolt
F_y	Yield stress of steel
F_u	Ultimate tensile strength of steel
FS	Safety factor
g	Total dead load of the building
g_k	k-th normalized constraint
h	Module depth

h	Elevation design variable
h'	Mutated elevation design variable
h_i	Initial height of the structure
I	Building importance factor
I_i	i-th size design variable
I_i'	mutated i-th size design variable
j	Total number of joints
k	Axial stiffness of member
K	Coefficient of effective buckling length
L	Length of member
L_i	Length of i-th member
L_s	Span length for the node havig maximum vertical deflection
n	Live load reduction factor
n_c	Total number of continuous design variables
n_{cn}	Total number of the constraints
n_d	Total number of discrete design variables
n_I	Total number of size design variables
n_{min}	Minimum number of members required for stability
$P(x)$	Penalty function
$P(t)$	Population of generation-t
P_{b-all}	Allowable tension capacity of bolts
P_d	Strategy parameter for discrete design variables
P_d'	Mutated strategy parameter for discrete design variables
P_I	Strategy parameter for size design variables
P_I'	Mutated strategy parameter for size design variables
P_{n-all}	Allowable compression capacity of nuts
P_s	Strategy parameter for support design variables
P_s'	Mutated strategy parameter for support design variables
q	Total live load of the building
r	Radius of gyration
R	Structural behaviour factor

S(T)	Spectrum coefficient
s_j	j-th support design variable
s_j’	mutated j-th support design variable
t	Number of generation
V_t	Total equivalent earthquake load
W	Total weight of the structure found by using live load reduction factor
W(x)	Objective function
w_k	available response for k-th constraint
w_{all}	allowable response for k-th constraint
α	Penalty coefficient
Δ_{max}	Maximum vertical deflection
γ_d	Learning rate for discrete strategy parameter
γ_I	Learning rate for size strategy parameter
γ_s	Learning rate for support strategy parameter
λ	Slenderness ratio
λ	Number of offspring
λ_i:	Slenderness ratio of i-th member
λ_p	Critical slenderness ratio
μ	Number of parents
$Ø_{x,y}$	Module angles
ψ	Geometric distribution parameter
ψ’	Mutated geometric distribution parameter
σ	Strategy parameter for continuous design variable
σ’	Mutated strategy parameter for continuous design variable
σ_{c-all}	allowable compressive stress
σ_{c-i}	maximum compressive stress on i-th member
σ_i	Initial value of elevation strategy parameter
σ_{t-all}	allowable tensile stress
σ_{t-i}	maximum tensile stress on i-th member
ρ	Unit weight of structural material
τ_c	Learning rate for continuous strategy parameter

τ_d Learning rate for geometric distribution parameter

τ_h Learning rate for elevation design variable

τ_I Learning rate for geometric distribution parameter in size mutation

CHAPTER 1

INTRODUCTION

1.1 General

Design of a structure has to provide some important requirements, which can be grouped as safety, serviceability, and economy. For safety of a structure, there are certain specifications and studies all around the world. Many architectural and client sourced requirements are also available according to type of the building for serviceability. However, there is no strict rule for the economy of the structures. This issue is completely left to contractor or owner's choice. Client, and contractors having lack of structural knowledge, care profit rather than total economy. Moreover, the limited sources of the world have to make us more sensitive to saving of these sources. An economic structure will reduce the usage of the raw material and be environmental friendly.

There is a big handicap that if a structure is more heavier, then it is safer. However, there is no direct relationship between the weight and safety of a structure. In some cases, making structure heavier, or using larger amount of structural material than required can badly affect the behavior of the structure. Moreover, making the structure lighter will also reduce the loads on the structure, especially loads due to earthquake, since they are directly related to mass of the structure. If the constraints of the structure like allowable stresses on the members, deflection limits, stability requirements, etc. are clearly defined according to certain provisions that previously determined, any optimization study can be done on the structure by remaining in constraint limits. Space trusses are appropriate structures for optimization.

Space trusses are three-dimensional structures having many different types separated according to type of members, connections, and materials. The most popular ones used in Turkey are double-layer grids, which have steel pipe sections for members, and have solid spheres for connections. Space trusses are generally used for covering areas having large spans without intermediate obstructions. Every piece of space trusses is prefabricated and so both manufacturing and assembling processes are very easy and fast. In addition, they are more economical compared to other structural systems like two-dimensional trusses, pre-tensioned concrete beams, etc. For all these reasons, space truss systems are preferred in many roofs and other types of structures having large spans.

Design of space trusses is a little different from conventional design applications. In conventional designs, parts of the structure are divided into some member groups, which have the same sections determined according to the most critical member of the group. However, section of every member in space trusses may be determined separately for maximum stresses on that member. This make optimization more efficient and important for space trusses. There are a few softwares used in industry for design of these types of structures. These softwares design structure for given load combinations by using some iterative techniques. However, these designs can be improved or optimized by using global optimization techniques.

1.2 Scope of the Thesis

In this study, evolution strategies method is adapted for optimization of space trusses. It is one of the stochastic optimization methods in the literature. It is inspired from biological evolution and natural selection phenomenon and simulates them to obtain optimum solution for structural design problems. In this thesis, double-layer grids and evolutionary algorithms are explained in details first, and then some available real-life structures, which are originally designed by Polarkon Steel

Structures Company (Polarkon SSC) are redesigned by using evolution strategies and finally results are presented.

In Chapter-2, double-layer grids are defined and a brief history is given. Advantages and disadvantages of these systems are mentioned. Parts of double-layer grid systems, including pipes, conics, nuts, bolts, spheres, supports, claddings, and purlins, are explained in details. In addition, analysis of space trusses, and load cases used in the analysis are defined clearly. Finally, design of each part is explained and formulated in this chapter.

Chapter-3 presents a literature survey for space trusses and their optimum design. The chapter refers to former important studies about issues, such as approximate analysis techniques for space trusses, progressive collapse, techniques to improve efficiency of structure, and nonlinear analysis of space trusses. Moreover, studies with different optimization techniques and different variations of these techniques are also given in this chapter.

Chapter-4 gives a comprehensive definition of evolution strategies used in this study. First, a brief history of evolution strategies, which shows the development stages, is overviewed. A complete definition for operators and other component of the evolution strategies are given. A simple algorithm is outlined for computer implementation.

In Chapter-5, adaptation of evolution strategies for optimization of space trusses and parameters used in this study are given. OFES, the software used for this optimization process is also introduced in this chapter.

In Chapter-6, six examples of real-life structures having 792, 1360, 1728, 2726, 3860, and 4412 number of members, are introduced and general properties of structures are summarized. Optimum solutions attained for each of these structures are presented. Optimization is based on design provisions imposed by Turkish specification, TS648 uzing size, elevation, and support design variables. The optimum designs are compared with those of FrameCAD software, used in the industry for design of such systems.

Chapter-7 is a conclusion chapter, which gives a brief summary of the present study. It also gives some recommendations for future works.

CHAPTER 2

DOUBLE-LAYER GRIDS

2.1 General

2.1.1 History

People have used different structural systems to span large open spaces since the pre-history times. From tents to domes, they have tried various techniques to provide larger column-free areas. The most common structural materials used in history were the stone and the concrete. They were strong enough under compressive stresses, but too weak under tensile forces. Lack of material that resists high-tension forces, made engineers to develop geometries preventing all members from tensile stresses. Arches and domes were very good trials, which suit these conditions. From 1250 BC to 1881 AC, many domes made of stones or any type of concrete had been built. They had spans up to 43 meters without any interior vertical support. However, the excessive weight of stone and the concrete made higher spans impossible. With the use of steel in structures, new systems have been developed. Steel is a lightweight material, which can resist both tensile and compressive stresses. This magnificent advantage made engineers use them in roof structures. Alexander Graham is the first scientist who developed the space grid systems (Chilton, 2000). Many structures were built in the beginning of 20^{th} century made of tubular steel members. However, space trusses became popular all over the world after the 1950's.

2.1.2 Definition

Space trusses are three-dimensional structures generally used to cover large spans without interior supports. Double-layer grids, which are formed by two parallel layers of top and bottom chords interconnected by inclined or vertical web members, are the most common type of the space trusses (Malla, 1996a) (See Figure-1). Members of space trusses are assumed to carry only axial loads. Members are connected to each other with numerous types of nodal connections. Any type of loads acting on structure has to be applied by these nodes, since loads applied along the length of the member quash the stability of system. To prevent this problem, purlins, which are directly connected to nodes, are used as a beam to support the load coming from roof covering. They take distributed loads along the length of member, and transfer them to space trusses through the nodes as point loads in gravity direction. However, in some examples of double-layer grids, the members of top chord can be designed to carry both axial and bending forces. Therefore top chord members can also be used as purlins to take loads directly from roof cladding (Cuoco, 1997). Space trusses are modular type of structures, which can be in square, rectangular, triangular or hexagonal forms. A rectangular type module of double-layer grids can be defined with following parameters (See Figure-2).

Figure – 1: Double-layer grids under construction

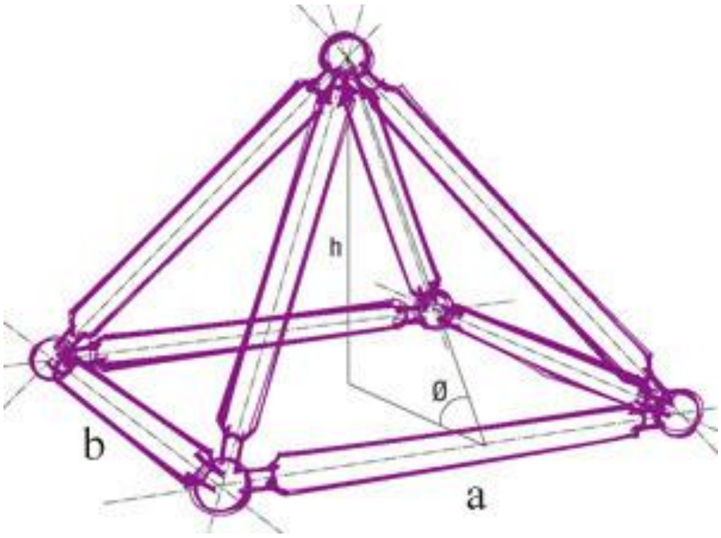

Figure – 2: A typical rectangular double-layer grids module

a: length of module in x-axis

b: length of module in y-axis

h: module depth

$Ø_{x,y}$: module angles

Module depth, h is directly concerned with the span of the structure. In the literature, depth to span ratio is given between 1/12.5 and 1/25 (Cuoco, 1997). However, when loads on the structure is higher or there are less number of columns that support the structure, this ratio can be increase up to 1/8, especially to remain in maximum deflection limits given in national or worldwide specifications.

2.1.3 Advantages

Double-layer grids have many advantages compared to other types of roof structures like two-dimensional trusses, reinforced or pre-stressed concrete beams. Most important difference in double-layer grids is three-dimensional load distribution in all parts of the structure. This causes a reduction in the weight of the structure by carrying loads with more members instead of single one, and thus maximum deflection along the structure is reduced. In addition, a separate design of every member in double-layer grids increases the efficiency in capacity use of members. Another important property of double-layer grids is that they are indeterminate systems. This means, the failure of one or a few number of members on the structure does not necessarily lead to collapse of structure completely (Chilton, 2000). By using higher safety margins in some critical members, the resistance of structure against to progressive collapse can be increased. Moreover, double-layer grids have large volume of free space between top and bottom chords. This permits to install any type of mechanical or electrical service systems continuously along the structure (See Figure-3). Additionally, every component of the double-layer grids is prefabricated, and so the quality of the each part is higher and the tolerance is lower compared to in-situ structures. The modular form also meets variety of architectural requirements easily.

Figure – 3: A gas pipe passing along the roof

2.1.4 Disadvantages

On the other side, double-layer grids have many disadvantages like vulnerability against fire and corrosion. Without taking any measures, the resistance of tubular structure to fire is negligible. Although there are some solutions like fire protective paints, none of them is economical due to large surface area of members. Corrosion is easier to deal with compared to fire. There are many economical solutions against corrosion in the market. Painting and galvanization are some of them. The rate of corrosion is also very low and foreseeable. However, fire is very sudden and can result in vital hazards.

2.1.5 Usage areas

Double-layer grids are generally used in large span roofs. Arch or dome shaped structures can easily be designed as double-layer grids. In addition, they can also be used as a vertical support like column or shear wall. The places where double-layer grids are used most frequently can be ordered as follows

- Factories,
- Warehouses,
- Plane hangars
- Terminal stations, parking lots
- Petrol stations
- Shopping malls
- Sports arenas, swimming pools, tennis courts
- Stadium roofs
- Cinemas, show centers, theaters
- Pedestrian bridges, etc.

2.1.6 Materials

Steel is the most generally used material in double-layer grids. It is light and economic for this type of structures. It is also easy to provide required accuracy in metals. Aluminum is another metal, which can give the same accuracy and it is lighter compared to steel. However, its modulus of elasticity is around 1/3 of the one that steel has. Therefore, maximum deflection in the structure may govern the design. To prevent excessive deflections, higher sections may be required and this may eliminate the lightness advantage of aluminum. Since steel is cheaper compared to aluminum, using steel in the structures is generally preferred all around the world. Timber and concrete are other materials used rarely in the double-layer grids.

2.2 Parts of Double-layer Grids

2.2.1 General

Double-layer grids can be classified in to two systems; in the first one, members carry both axial load and moments and in the second one members carry only axial load. It depends on how the members are connected to each other. It is also important if the members are connected to each other eccentrically or not. In roof type structures, the connection of cladding to space truss determines the behavior of the structure. Cladding is used on the structural system to cover ceiling. Another function of cladding is to transmit load to structural system. Cladding can be mounted directly on top chord members of structure or purlins are used for this load transmission. Using purlins that directly connected to nodal elements converts distributed loads of roof to nodal loads in gravitational direction. Therefore, top chord members are designed for only axial loads in this case.

Pipes, boxes and double angles are most general sections for members of the double-layer grids. Nodal members can be fabricated as, flat plates, pressed plates, castings, forgings or extrusions (Cuoco, 1997). Around the world, every company has its own style to select the type of members and connections. In this study, pipe sections are used for members, which are acted by only axial loads and solid spheres are used for nodes. Other connection elements are conics, nuts, and bolts, which provide connection between pipes and spheres. Figure-4 shows a complete member of double-layer grids, which is generally used in structures designed by Polarkon SSC.

This one is also the most common type used in Turkey due to ease in manufacturing and assembling.

Figure – 4: A typical double-layer grids member

2.2.2 Pipes

Pipes are the main part of space trusses, which contribute most to total weight of the structure. Under tensile stresses with no bending, the shape of cross-section is not important. Strength of members is determined by cross-sectional area of profiles in this case. However, in case of compressive stresses the geometry of section gains importance due to buckling effect. In this case, member fails about an axis having critical moment of inertia. Since the members of double-layer grids are sensible to compressive forces, the use of sections like pipes, which have equal moment of inertia in all directions, is a very suitable selection. Therefore, many contractors prefer using pipe sections for all top chord, bottom chord, and web members to make lighter and economical structures. Polarkon SSC also uses hot rolled steel pipe sections.

Table 2.1 shows outer diameters and thicknesses of the pipe sections used in examples of this study. Different combinations of these sections were used in each original design according to availability of sections in the stock of the company and requirements of the project under consideration. The use of all the sections in a project has certain shortcomings. High number of different sections makes production harder due to different formwork applications and assembling higher number of different pipe types in the construction site is more time-consuming task. For each project, an efficient group of these sections is chosen. Each example given in this study includes profile list used in original design. Exactly the same profile lists are also used in optimum designs.

Table 2.1: Outer Diameter and Thicknesses of Pipe Sections

Outer Diameter (mm)	Thickness (mm)	Grade
42.4	2.50	St37
48.3	2.50	St37
48.3	3.00	St37
48.3	3.25	St37
48.3	3.50	St37
60.3	2.50	St37
60.3	3.00	St37
60.3	3.40	St37
60.3	3.50	St37
60.3	3.65	St37
76.1	2.90	St37
76.1	3.40	St37
76.1	3.50	St37
76.1	3.65	St37
88.9	3.00	St37
88.9	3.50	St37
88.9	3.76	St37
88.9	4.00	St37
88.9	4.05	St37
88.9	4.50	St37
88.9	5.00	St37
114.3	4.05	St37
114.3	4.50	St37
114.3	5.00	St37
139.7	4.00	St37
139.7	4.50	St37
139.7	5.00	St37
139.7	6.00	St37
159.0	4.50	St37
219.1	4.50	St37
219.1	4.50	St52
219.1	6.00	St37
219.1	6.00	St52
219.1	7.00	St52
219.1	11.00	St52

2.2.3 Conics

Conics are intermediate members, which connect pipes to nuts. A small portion of the conic remains in the pipe and they are connected to each other with lap fillet welding. Conics provide an appropriate surface for connection of nuts to the member by reducing outer diameter of the member linearly. Outer diameters of conics have to be smaller than inner diameter of pipes, but not more than a predefined tolerance, which is about 1.0 mm. The diameter of surface connected to nut has also to be larger than the outside diameter of nuts. According to chosen pipe and nut sections, an appropriate conic is selected. Polarkon SSC uses conics, which are made of hot forged steel with quality of C1020.

2.2.4 Nuts

In double-layer grids members, nuts have the function of screwing bolts to solid spheres by wrench. The head of bolt remains in the conic, so they cannot be screwed directly from head. Instead of this, nuts are tied to bolts by a screw and so bolts can be turned with nuts at the same time (See Figure-5). This provides easier assembling in the site.

Figure – 5: Nuts tied to bolts by a screw

Inner diameter of nuts is determined according to diameter of bolts used in the member. Outer diameter of nut depends on compression force acting on member, since compressive strength of nuts is linearly proportional to their own surface area. Required surface area is provided by selecting an appropriate outer diameter. Table 2.2 shows outer and inner diameters and allowable compression stresses of nuts used in designs of Polarkon SSC.

Table 2.2: Outer and Inner Diameters and Allowable Compression Stresses of Nuts

Outer Diameter (mm)	**Inner diameter (mm)**	**Allowable compressive stress(t/cm^2)**
19	13	1.8
27	18	1.8
30	22	1.8
36	22	1.8
41	22	1.8
46	22	1.8
41	29	1.8
46	29	1.8
55	29	1.8
60	29	1.8
65	29	1.8
46	32	1.8
50	32	1.8
50	33	1.8
60	35	1.8
55	35	1.8
75	38	1.8
60	38	1.8
75	41	1.8
75	44	1.8
75	44	1.8
65	44	1.8
70	44	1.8
99	50	1.8
75	50	1.8
80	50	1.8
99	64	1.8
99	66	1.8

2.2.5 Bolts

Function of bolts in double-layer grid members is connecting all pipes, conics, and nuts of the members to spherical nodes. They are pinned to nuts, and by wrenching nuts, they are screwed into spheres. Total length of bolts is not important. However, the length remains in the sphere determines the anchorage strength, and it has to provide tensile strength of the bolt. The laboratory tests made by Polarkon SSC shows that minimum anchorage length of bolt has to be larger than its diameter. The diameter of the bolt is determined according to tension force acting on the member.

Table 2.3: Diameters and Quality of Bolts

Diameter (mm)	Quality
12	8.8
12	10.9
16	8.8
16	10.9
20	8.8
20	10.9
27	8.8
27	10.9
30	8.8
30	10.9
33	8.8
33	10.9
36	8.8
36	10.9
39	8.8
39	10.9
42	8.8
42	10.9
48	8.8
48	10.9
56	10.9
60	8.8
60	10.9
64	10.9

Polarkon SSC uses high strength bolts, which have grade of 8.8 or 10.9. Table 2.3 shows diameters and quality of bolts used by Polarkon SSC.

2.2.6 Spheres

Spheres are nodal elements of double-layer grids. They are solid members, which have the second largest contribution to total weight of structure. They connect members to each other with the help of bolts. Eight members are connected to a sphere in a typical rectangular double-layer grids module. On the other hand, ten more holes can be drilled in special cases. Not only structural members, but also some other items for service and cladding can be attached to nodes with these additional holes. In typical double-layer grids, additional holes are made at top of spheres in every top chord nodes to support purlins, and at the bottom of spheres in every bottom chord nodes for service equipments.

In Polarkon SSC, spheres having diameters of 60, 75, 90, 110, 132, 154, 190, and 240 mm diameters are used generally. They are made from hot forged steel with C 1040 quality. Solid spheres have unit costs (cost/kg) increasing with diameter. With increased diameter, both weight and cost of spheres are multiplied. Therefore, dimension of the spheres have to be kept as smaller as possible. Relative angles between members connected to sphere directly affect the diameter of the spheres. Narrow angles results to intersection of members connected to same sphere, so diameter of the sphere is needed to be increased. So geometry of the structure must be prepared cautiously to prevent narrow angles between members.

2.2.7 Supports

Double-layer grids can be supported on reinforced concrete columns, shear walls, steel columns, or directly on the foundation. For all these cases, general properties of the supports are same. They are made of one steel plate, a 10~20 cm steel pipe on this plate, and a conic welded to top of pipe. The sphere at support node is welded to this conic.

Supports are assumed as restrained in gravity direction. Restraints in horizontal directions can be removed or restored as required by using roller or pinned supports respectively. In restrained cases, the horizontal stiffness of the substructure has to be considered. Roof has to be analyzed with substructure simultaneously, or horizontal stiffness values of substructure have to be assigned on support nodes of space truss as springs in the analysis model.

Area to number of supports ratio is an important parameter in double-layer grids. Increasing of this ratio also results in an increase in the weight of the structure due to excessive shear forces around the supports. Increasing of the span also causes an increase in both weight and maximum vertical deflection of the roof. Insufficient number of supports also causes an increase in reaction forces acting on the supports. These reaction forces can result in excessive moments at the base of columns and this outcomes higher sections of columns and foundations in the substructure.

2.2.8 Cladding and purlins

Cladding is the outer part of the roof, which takes all wind, and snow loads and transmits them to structural members. There are various type of claddings used in the market, which changes according to needs and budget of the project. The most used ones in Turkey are trapeze sheets and sandwich panels. These are steel profiles having thickness generally changing between 0.3 and 1.0 mm. Their moment of inertia in strong axis is high but negligible in weak direction, so they are supported by purlins in weak direction. Since no load is permitted along the length of pipes, purlins can only be mounted on the top nodes of double-layer grids. Figure-6 shows a detail of cladding connected to purlins by bolts. Type and thickness of cladding determine the span length of the purlins supporting cladding. The module width perpendicular to purlins has to be determined according to cladding requirements. In case that module width cannot be reduced, secondary purlins mounted on primary purlins can be used with required span lengths.

Purlins and cladding are relevant to architectural requirements, and they are not included in optimization process in this study.

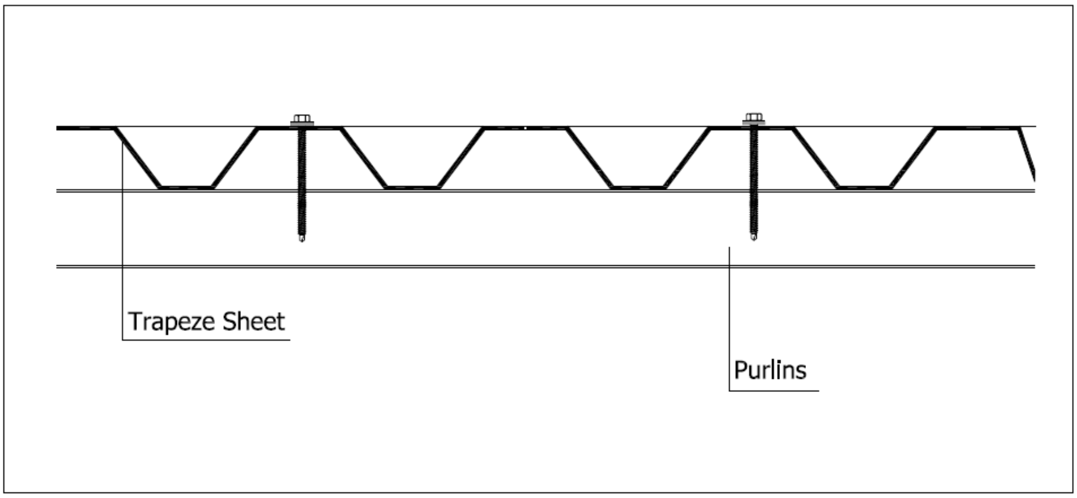

Figure – 6: A cladding detail on purlins

2.3 Analysis and Design of Double-Layer Grids

2.3.1 Analysis

2.3.1.1 General

Double-layer grids are composed of nodal and tubular elements, which are designed only for axial tension and compression loads since spherical nodal elements are assumed not to transmit any moment and rotation. In finite element analysis method, members of space trusses have a 6 x 6 global stiffness matrix having three degrees of freedom at start and three degrees of freedom at the end of the member. By equalizing the force matrix with the product of deflection and stiffness matrix, the deflections of the nodes and the forces along the members are obtained. Stiffness of the members in axial direction is obtained as follows,

$$k = \frac{EA}{L} \qquad (2.1)$$

A is the cross-sectional area of the member, and L is the length of the member. E is young's modulus, which is assumed as 2100 t/cm^2 for structural steel according to TS648.

2.3.1.2 Loads

Roofs, especially space roofs, are special structures compared to other components of buildings. Self-weights of space trusses are very light, sometimes negligible compared to total load, and they may have very large spans. Therefore, each load case may be critical and has to be considered in the analysis and design. The loads acting on double-layer grids may be static or dynamic. Temperature effect has also to be considered as a load case in the analysis. Loads, having periods much larger than the natural period of the structure, are defined as **static loads** (Malla et al., 1996a). Static loads acting on double-layer grids can be ordered as follows,

- Self-weight of structure,
- Live loads,
- Weight of purlins,
- Weight of cladding,
- Service and various equipment loads,
- Snow and/or ice load,
- Rain (collected water) load,
- Loads due to misalignment of members
- Differential settlement of supports, etc.

The quantity of these loads changes according to a variety of parameters based on location and functionality of the structure. However, it is hard to determine stresses on members due to last two items. They are generally ignored or handled by using a safety factor. To reduce misalignment, tolerance is defined as between 0.5 and 1.0 mm for double-layer grids in the literature. In the conventional design of double-layer grid systems obtained with FrameCAD, stresses owing to imperfections were ignored by keeping errors in length of members less than 1.0 mm. Consequently, additional stresses due to imperfect fit of members were also disregarded in optimum designs.

In most type of the space roofs, cladding has a minimum slope to make water flow to sides and then it is transferred to water systems by gutters along the sides of roof. So that, rain load is not a common load case in double-layer grids.

Most common loads acting on space trusses are weight of purlins and cladding, service loads, and snow load. Weights of purlins and cladding are generally approximated, and they are usually assigned on top level of the roof. Service loads are defined by user according to tools hanged on the roof. These tools can be hanged both on top or bottom level nodes of the structure. Cat walks, any installation for cabling, air conditioning, any tools for cooling and heating are included in service loads. Snow load is generally governs the design results since it is usually larger as amount compared to other loads in gravity direction. It changes with the climate in the location of the construction. TS498 gives minimum amounts for snow load for Turkish cities. However, these amounts are generally increased in the design examples considered in this study to comply with non-optimum original design considerations.

Another load type acting on double-layer grids is **dynamic loads**. Basic dynamic loads acting double-layer grids are as follows,

- Earthquake,
- Wind,
- Vibrations of vehicles or machine,
- Impacts or blasts, etc.

An important difference between static and dynamic analysis is that the dynamic analysis considers the inertia effects leading to equation of motion (Malla et al., 1996b). There are several methods for dynamic analysis of the structures like double-layer grids. Equivalent static load method is the one of the most common and easiest method amongst them. In this way, dynamic loads are converted to approximate static loads, which create the most critical forces on the structure. In some specific structures, modal analysis and time history analysis can be required. However, they are not commonly preferred for space trusses. In scope of this study, all examples were solved with equivalent static load method similar to their conventional designs.

There are many studies, which explain the effects of vibration, impacts, or blasts on the structures in the literature. On the other hand, none of them has been taken into account in current design practice. Accordingly, these load cases are not considered in this study neither.

Earthquake and wind loads are the most common dynamic load cases used in the analysis of double-layer grids. Both of these dynamic load cases are converted to equivalent static loads in this study. Evaluations of equivalent wind and earthquake forces are explained in the following sections.

2.3.1.3 Evaluation of wind load

Quantity of equivalent wind load is available in TS498 as an area load. It changes according to height of the structure from foundation level. Wind makes pushing effect on the surface of horizontal double-layer grid systems, which wind directly hit, and suction on all other surfaces of the structure as equal to half of the pushing amount. According to TS498, these amounts are given as 0.8q for pushing and 0.4q for suction (See Figure-7). *q* is the area load given for different ranges of building height in the same specification. On top of the roof, this amount changes with the angle that top roof surface makes with the horizontal plane. If substructure that support roof has open surfaces in sides, then pushing effect also occurs on the top of the roof from bottom to top. In this case, wind creates 1.2q load on the top of structure in opposite direction of gravity if the angle of the roof with the horizontal plane is zero. It may be critical, since bottom layer members, which generally designed for tension, will take also compression loads and will be introduced by buckling effect.

After determining equivalent wind loads as area loads, they have to be converted to point loads. Tributary areas can be evaluated for each node on the loaded surface and point load of each node can be found by multiplying these areas with the area load given in the specification. As an approximate way, area load can be multiplied with the total surface area acted by wind and then, it is divided by the number of the nodes on that surface. The approximate approach is more convenient to use, and does not cause significant errors.

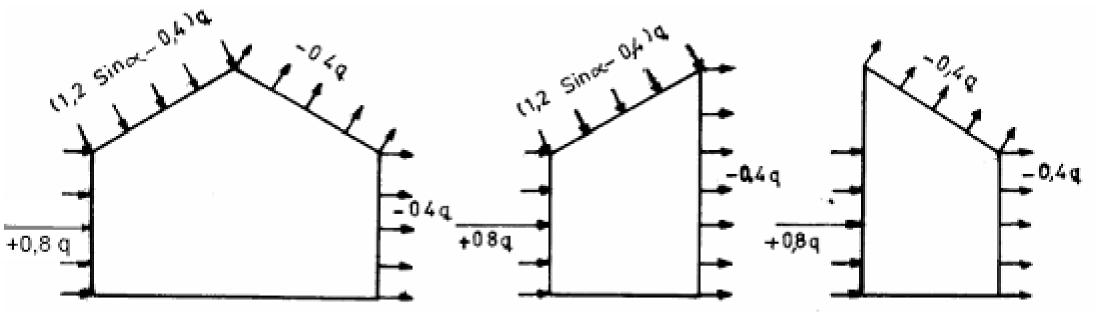

Figure – 7: Distribution of wind force (TS498)

<u>*2.3.1.4 Equivalent static load for earthquake*</u>

In this section, evaluation of earthquake excitation as equivalent static load is explained in detail. The calculation given here are based on 2007 Turkish Earthquake Specification. Actually, this specification is used for buildings and does not cover large span roofs. Nevertheless, it is not an improper approach as far as calculation of horizontal component of the earthquake is concerned, since double-layer grids behave like slabs in horizontal direction. In the example problems considered in this study, the effect of earthquake in vertical direction is ignored. It should be emphasized that, in many double-layer grids, the first natural period of the structure occurs in vertical direction. This case may be critical when excitation occurs from bottom to top, since members in bottom layer of the structure are loaded in compression instead of tension. Nevertheless, to take the same design considerations, vertical earthquake excitation is also ignored in their optimum design. However, negligence of vertical component is not suggested in many technical papers.

According to Turkish Earthquake Code, total horizontal equivalent earthquake load is given by the equation (2.2).

$$V_t = \frac{WA(T_1)}{R(T_1)} \qquad (2.2)$$

In this equation, *W* defines the total weight of the structure found by using live load reduction factor. $A(T_1)$ is the spectral acceleration coefficient for natural period of the structure. $R(T_1)$ is the structural behaviourfactor of the structural system. *W* can be found by the following formula,

$$W = g + nq \qquad (2.3)$$

In eq. (2.3), *g* is total dead weight on the building. *q* is total live load on the building, and *n* is live load participation factor. For space trusses, g can be taken as sum of the own weight of structure, weight of cladding and purlins, and service loads. If roof is in a snowy place, using only snow load is enough for live load. On the other hand, if there is no snow, an amount of live load can be taken for any possible activity on the roof. The *n* value is given as 0.3 for snow in the code.

A(T) is given in Turkish Earthquake Code as follows,

$$A(T) = A_0 I S(T) \qquad (2.4)$$

Where, A_0 is effective earth acceleration coefficient, which is given according to earthquake region in Table 2.2 of the code. I is building importance factor, which is again given for usage aim of the building in Table 2.3 in the code. *S(T)* is spectrum coefficient. This value has to be determined according to natural period of the structure, and ground type of the building. With these known parameters, minimum S(T) value can be found from spectrum given in any earthquake code. Instead of calculating all these values, in conventional solutions of the design examples considered in this study, S(T) value was directly taken as 2.5, which is the maximum value given in the spectrums. This assumption used for horizontal component of earthquake does not cause an important change in weight of structure, since structure is very rigid in horizontal directions. However, it will result in high stresses on the members of substructure.

Another parameter given in eq. (2.2) is $R(T_1)$. This value is given in Turkish Earthquake code according to type and ductility of the structural system. The code gives $R(T_1)$ value as 4 for the buildings that carry all of the earthquake loads with the pin-connected columns at the top and have normal ductility. The definition given for this item is close to buildings having space roofs. However, this value is given for design of substructure, not the space roof. Therefore, none of them is exactly appropriate for space roofs actually. It is known that, members of double-layer grids are under risk of brittle failure due to buckling effect. In addition, the energy consumption is limited at connections, since there is no bending. For these reasons, it is rational not to use large values for $R(T_1)$. In the conventional solutions of the design examples considered in this study, it is taken as 3 or 5.

Finally, with known V_t, point loads that acts on every node of the structure can be found by dividing V_t by total number of the nodes. In the analysis model, these loads are assigned to nodes both in x and y-direction.

2.3.1.5 Load combinations

Combinations are selected to consider every critical case for the structure. There are different combination lists used for each example given in this study. Exactly same combinations are used with the ones used in conventional designs. Combination lists for each example are shown in the relevant chapter.

2.3.2 Design of Pipes

In double-layer grids, pipes are members, which carry both axial tension and compression loads. In compression design, buckling effect is also checked. Building Code of Steel Structures of Turkish Standards, TS648 is used to check members for both tension and compression forces as parallel to original non-optimum designs.

2.3.2.1 Design for tension

Allowable tensile stress, $F_{t\text{-}all}$ is given as the minimum value obtained from following equation,

$$F_{t-all} = \min\left\{\left(0.6 \times \mathrm{F}_y\right), \left(0.5 \times \mathrm{F}_u\right)\right\} \qquad (2.5)$$

F_y is the yield stress of steel, and F_u is the ultimate tensile strength of steel. Actually, when equations given above are evaluated, it is seen that both specifications give exactly the same allowable strength for tension members of double-layer grids.

2.3.2.2 Design for compression

Doubly symmetric members under axial compression fail only under flexural buckling. In TS648, allowable compressive stress, $F_{c\text{-}all}$ is defined as given in equation (2.8) according to the ratio of slenderness ratio, λ to critical slenderness ratio, λ_p.

$$\lambda = \frac{KL}{r} \tag{2.6}$$

$$\lambda_p = \sqrt{\frac{2\pi^2 E}{F_y}} \tag{2.7}$$

$$\mathrm{F_{c-all}} = \begin{cases} \dfrac{\left[1 - 0.5\left(\dfrac{\lambda}{\lambda_p}\right)\right] F_y}{n}, & \lambda < \lambda\mathrm{p} \\ \dfrac{2\pi^2 E}{5\lambda^2}, & \lambda \geq \lambda\mathrm{p} \end{cases} \tag{2.8}$$

Formulas for slenderness ratio and critical slenderness ratio are given in equation (2.6) and (2.7) respectively. *n* defines factor safety, which is evaluated as follows,

$$n = \begin{cases} 1.67, & \lambda < 20 \\ 1.5 + 1.2\dfrac{\lambda}{\lambda_\mathrm{p}} - 0.2\left(\dfrac{\lambda}{\lambda_\mathrm{p}}\right)^3, & 20 \leq \lambda < \lambda\mathrm{p} \\ 2.5, & \lambda \geq \lambda\mathrm{p} \end{cases} \tag{2.9}$$

2.3.3 Design of Bolts

Bolts are designed only for tension in double-layer grid systems. Their allowable tensile strength is determined by dividing their ultimate tensile strength by a safety factor, which can be taken 0.5 as given in TS648 for tension members. However, Polarkon Steel Structures Co. engineers prefer a safety factor, which equals 0.4 to be on safe side.

In this study, high strength bolts with steel grades of 8.8 and 10.9 having ultimate tensile strength of 8.0 and 10.0 t/cm^2, respectively are used similar to conventional solutions of the design examples.

Critical area of a bolt is accepted as minimum of the core and the net area of the bolt shank. Bolts used in double-layer grid systems have a hole, which is changing between 4 and 6 mm. This hole is to pin the bolt to nut and it has to be considered in net area calculation as shown in Equation 2.10. Computation of core area is also given in Equation 2.11. After this, allowable tension force of the bolt, $P_{b\text{-}all}$ can be found by Equation 2.13.

Table 2.4: Allowable Tension Capacities of Bolts

Bolt	Core Area (cm^2)	Pin Diameter (mm)	Net Area (cm^2)	Quality	Critical Area (cm^2)	Safety Factor	Allowable tensile force (ton)
M12	0.84	4	0.65	8.8	0.65	0.40	2.08
M12	0.84	4	0.65	10.9	0.65	0.40	2.60
M16	1.49	4	1.37	8.8	1.37	0.40	4.39
M16	1.49	4	1.37	10.9	1.37	0.40	5.48
M20	2.32	5	2.14	8.8	2.14	0.40	6.85
M20	2.32	5	2.14	10.9	2.14	0.40	8.57
M27	4.23	5	4.38	8.8	4.23	0.40	13.55
M27	4.23	5	4.38	10.9	4.23	0.40	16.94
M30	5.23	5	5.57	8.8	5.23	0.40	16.73
M30	5.23	5	5.57	10.9	5.23	0.40	20.91
M33	6.33	6	6.57	8.8	6.33	0.40	20.24
M33	6.33	6	6.57	10.9	6.33	0.40	25.30
M36	7.53	6	8.02	8.8	7.53	0.40	24.09
M36	7.53	6	8.02	10.9	7.53	0.40	30.11
M39	8.84	6	9.61	8.8	8.84	0.40	28.27
M39	8.84	6	9.61	10.9	8.84	0.40	35.34
M42	10.25	6	11.33	8.8	10.25	0.40	32.79
M42	10.25	6	11.33	10.9	10.25	0.40	40.99
M48	13.38	6	15.22	8.8	13.38	0.40	42.83
M48	13.38	6	15.22	10.9	13.38	0.40	53.53
M52	18.22	6	21.27	10.9	18.22	0.40	72.87
M60	20.91	6	24.67	8.8	20.91	0.40	66.92
M60	20.91	6	24.67	10.9	20.91	0.40	83.65
M64	23.79	6	28.33	10.9	23.79	0.40	95.17

$$A_{net} = \frac{\pi d^2}{4} - d_h d \qquad (2.10)$$

$$A_{core} = \frac{\pi(0.86d)^2}{4} \quad (2.11)$$

$$A_{cr} = min(A_{core}, A_{net}) \quad (2.12)$$

$$P_{b-all} = A_{cr}F_{ult}FS \quad (2.13)$$

In these equations, d is diameter of bolt, d_h is diameter of pin hole, F_{ult} is ultimate strength of bolt, and FS is the safety factor. Allowable tension capacities of bolts used in the examples are as shown in Table 2.4.

2.3.4 Design of Nuts

Nuts are compression members in double-layer grid systems. However, they do not buckle due to their short lengths. The compression capacity of nuts is found by multiplying critical area of nut with allowable compression stress. Critical area of nuts is calculated by Equation 2.14. Allowable compression force, P_{n-all}, is given in Equation 2.15.

$$A_{cr} = 0.866{d_o}^2 - \frac{\pi {d_i}^2}{4} - (d_o - d_i)d_h \quad (2.14)$$

$$P_{n-all} = A_{cr}F_{all} \quad (2.15)$$

F_{all} is taken as 1.8 t/cm^2 in the design examples. d_o is outer diameter; d_i is inner diameter of the nut, and d_h is the diameter of pin hole. Allowable compression capacities of nuts used in the design examples are as shown in Table 2.5.

2.3.5 Design of Spheres

In double-layer grids, spheres are nodal elements, which provide connection of pipe members. Spheres are solid steel members, which have toothed round holes for every member it connects. The diameter of a hole depends on the bolt diameter used in a connected member.

Table 2.5: Allowable Compression Capacities of Nuts

Nut Outer Diameter (mm)	Nut Outer Diameter (mm)	Pin Hole Diameter(mm)	Area (cm^2)	σ_{em} (t/cm^2)	Compression capacity (t)
19	13	4	1.56	1.8	**2.81**
27	18	4	3.41	1.8	**6.14**
30	22	5	3.59	1.8	**6.47**
36	22	5	6.72	1.8	**12.10**
41	22	5	9.81	1.8	**17.65**
41	29	5	7.35	1.8	**13.23**
46	22	5	13.32	1.8	**23.98**
46	29	5	10.87	1.8	**19.56**
46	32	5	9.58	1.8	**17.25**
55	29	5	18.29	1.8	**32.92**
60	29	5	23.02	1.8	**41.44**
60	35	6	20.05	1.8	**36.10**
65	29	6	27.82	1.8	**50.08**
75	38	6	35.15	1.8	**63.27**
75	41	6	33.47	1.8	**60.25**
75	44	6	31.65	1.8	**56.96**
75	44	6	31.65	1.8	**56.96**
99	50	6	62.30	1.8	**112.14**
99	64	6	50.61	1.8	**91.09**
99	66	6	48.68	1.8	**87.63**
50	32	5	12.71	1.8	**22.87**
50	33	6	12.08	1.8	**21.74**
55	35	6	15.38	1.8	**27.68**
60	38	6	18.51	1.8	**33.33**
65	44	6	20.12	1.8	**36.22**
70	44	6	25.67	1.8	**46.20**
75	50	6	27.58	1.8	**49.64**
80	50	6	33.99	1.8	**61.18**

There is no statically required design criterion for solid spheres. Being solid makes them excessively strong compared to all other parts of the member. Diameter of spheres is determined by checking if there is any intersection between bolts, nuts, and pipes of the members that are connected to same sphere. Any diameter of spheres can be used as long as it satisfies the following four geometric constraints between the connected members.

$$\mathrm{c} - \mathrm{a}_1 - \mathrm{b}_1 \geq 0 \tag{2.16}$$

$$\mathrm{c} - \mathrm{a}_2 - \mathrm{b}_2 \geq 0 \tag{2.17}$$

$$\mathrm{c} - \mathrm{a}_3 - \mathrm{b}_3 \geq 0 \tag{2.18}$$

$$\mathrm{d}_\mathrm{c} \geq 2 \times d_{max} \tag{2.19}$$

In these equations, c is the angle between two axes of members. a_1 and b_1 are the angles between the member axes and the side of bolt, a_2 and b_2 are the angles between member axes and the side of nuts, and a_3 and b_3 are the angles between the member axes and the side of conics respectively as showed in Figure-8. Diameter of sphere must be greater than two times of maximum bolt diameter connecting to node, d_{max} to prevent intersection of bolts in the center of sphere.

2.3.6 Design of Conics

There are no statically required design criteria for conics, since they have safer section properties compared to connected pipe and nut. According to inner diameter of pipe and outer diameter of nut, an appropriate conic is selected.

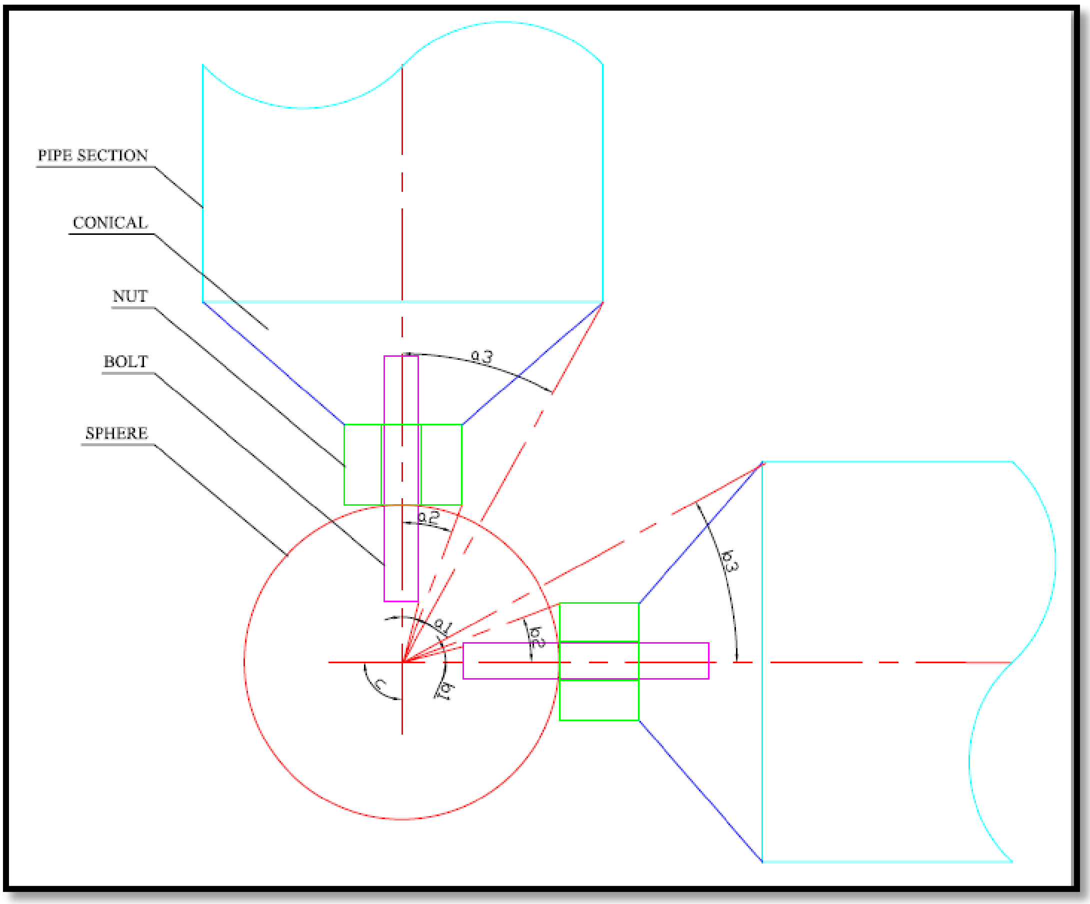

Figure – 8: a_1, a_2, a_3, b_1, b_2, b_3 angles for two sequent members

CHAPTER 3

LITERATURE SURVEY

Studies on the analysis and design of space trusses have been very common, especially for last two decades. Some of them includes the same subject with present work, which contains, analysis, design and optimization of space trusses. Moreover, there are some other studies, which focus on important details not directly related to present work.

A general overview for double-layer grids is given by Malla et al. (1996a, 1996b) with two papers. General information and important references about loading, linear and nonlinear analyzing methods, thermal and dynamic analysis, progressive collapse and optimization of space trusses are given in detail. Later, Cuoco (1997) published a more comprehensive study about these subjects as a booklet. Chilton (2000) is another important source to obtain general information about space trusses.

Approximate analysis of space trusses is another important subject. It is preferred in optimization of space trusses having large number of members, since it is very time consuming to analyze these structures. Kaveh et al. (2001) presented an approximate analysis method for double-layer grids by using back propagation neural network. By using some input parameters like span length, height of the structure, etc, an approximate design can be done. Moghadas et al. (2008) estimated maximum deflection of the structure by using similar inputs. Papadrakakis et al. (1998), and Lagaros, N.D. et al. (2005) presented a study, which uses neural network in structural analyze stage of large structures' optimization by using evolution strategies. Greco et al. (2006) presented a new geometric nonlinear formulation for static problems involving space trusses. The proposed formulation is based on the finite element method. It is simple and yields results with negligible error.

Smith (1984) underlined how linear elastic analysis is insufficient to determine the behavior of space trusses accurately and presents a nonlinear stepwise linearization analysis method, which does not require the repeating updating of structural stiffness matrix.

After the collapse of some space trusses, many papers on progressive collapse of these structures were published and inelastic analysis gained importance to find safety margin of structures against progressive collapse. Blandford (1996) presented the details of a nonlinear analysis program for space trusses. Moreover, Murtha-Smith (1988) advised that compression members and diagonals along and adjacent to the line between columns should be overdesigned, particularly in the middle half of the span for protection from progressive collapse.

Various techniques are studied to improve efficiency and reduce the cost of double-layer grids. Dehdoshti et al. (1996) discussed the method of post-tensioning of space trusses, which makes it possible to form dome-shaped space trusses of interesting architectural shapes by using members of same length. Liew et al. (2006) presented the cable-strut structure, which is a special form of space frame system. By using high-strength cables for web and bottom chord members, which are under tensile stresses, total weight of structure and complexity at joints can be reduced. Quirant et al. (2003) presented another type of space trusses, tensegrity systems, which include a discontinuous set of compressed components inside continuous tension members. In addition, Symons et al. (2005a & 2005b) presented Kagome double-layer grids, and analysis, performance, and effects of imperfections on the structure are shown in details.

Removing of some diagonal members to distribute load more uniformly in the structure is another popular subject for space trusses. Tabatabei et al. (1993), and

Gargari (1993) demonstrated the effects of removing diagonals and introduce a method to obtain an optimum increase in load carrying capacity of space trusses.

There are many papers about optimization of two-dimensional trusses in the literature. However, research on space trusses having a certain number of members is rather limited. Optimization of space trusses is a developing subject, which gains importance with increasing capacity of computers. Various optimization methods have been developed for this issue in the last decade. Most popular ones are metaheuristic or global optimization methods.

Back et al. (1996) gave general information about evolution algorithms and underlines the importance of using self-adaptive strategy parameters in evolution strategies and evolutionary programming. Winter et al. (1996) presented a comprehensive study about computational implementation of evolution strategies. Thierauf et al. (1997) presented a method named as parallel evolution strategy, which provides optimization of both continuous and discrete variables at the same time. Gutkowski et al. (2001) introduced controlled mutation technique into evolution strategies in which mutation depends on the minimum and maximum stresses on the members. Lagaros et al. (2002) showed a hybrid methodology formed by combined genetic algorithms and evolution strategies, which is tested in large-scale structures. Rajasekaran et al. (2004) introduced a multilevel optimization approach in evolution strategies to reduce design space in the optimization of space trusses. Baumann et al. (2005) used topology optimization to build-up structures starting from simple initial configurations by using simulated annealing, evolutionary algorithms, and random cost method. Hasançebi (2008) presented a study, which demonstrates the use of adaptive evolution strategies in the optimization of large-scale structures including examples of 26-storey and 942-bar truss problems. Hasançebi et al. (2009) presented a comparison of metaheuristic search techniques in the optimization of pin-jointed structures. It reveals that simulated annealing and evolutionary strategies give better solutions compared to other methods.

CHAPTER 4

EVOLUTION STRATEGIES

4.1 Introduction

4.1.1 General

Optimization of structural systems is a research field, which is under continuous development. New optimization techniques, in addition to extensions or modifications of the existing ones are proposed incessantly. Optimization methods can be grouped under three main branches. One of them is named as **enumerative techniques**. These techniques reache optimum solution by searching every possible point in design area. They can be applicable for small structures. However, they are not appropriate methods for large structures under today's technology. Although implementation of these techniques are very simple, a large amount of computational effort is required. Another optimization method is **calculus-based techniques**, which are also called as hill climbing techniques. These techniques try to reach optimum solution by using derivative information of the variables. Their shortcoming is that they easily get stuck in local optima and they require a continuous design space for implementation. Therefore, they are not suitable for optimization of large structures. The third methods are called as **stochastic or global optimization techniques**. There are many of these methods available in the literature. Simulated annealing, evolutionary algorithms, tabu search, harmony search, swarm-based optimization techniques are some of them. The common feature in these methods is that they avoid a gradient based search and use randomized operators rather than deterministic ones.

Evolution algorithms are inspired by Darwin’s evolution theorem and natural selection concept of this theorem. Survival of the fittest rule is simulated for a population of different solutions or individuals of the problem. First, an initial set of solutions are randomly generated from a given set or range of values according to type of variables. After this initialization, algorithm continues with evaluation of these individuals for fitness. Then, individuals are recombined and mutated to create new offspring consistent with some predefined parameters. Offspring are again evaluated according to objective function of the optimization. Individuals having best fitting values survive to form new generations and the others are removed from population. By repeating this process, optimum or near-optimum solution will be reached.

Evolution algorithms have three types named as, evolutionary programming(EP), genetic algorithms(GAs), and evolution strategies(ESs). They are all same basically, but there are certain differences in using operators and representation of individuals. Mutation is the main operator in ESs and EP, however it is secondary in GAs. On the other hand, while recombination is the main operator in GAs, it is secondary in ESs and not applicable in EP. Selection operator is probabilistic for GAs and EP, nevertheless it is deterministic in ESs. Finally, while individuals are represented by real variables in ESs and EP, binary coding is used in GAs.

Evolution strategies generally converge the optimum solution for double-layer grid systems in fewer time compared to other evolutionary algorithm techniques. Also in this study, ESs is used to find optimum solutions.

4.1.2 History

Evolution strategies are first developed by Igno Rechenberg and Hans Paul Schwefel in 1960’s (Back, 1996). This original version is a (1+1)-ES type that means it has only one parent and one offspring in population. Recombination operator is not applicable in this version since there is no couple in the population. One parent is mutated, and the better one of the mutated and parent individual survives. In 1973, I. Rochenberg develops (μ+1)-ES. In this version, there are $\mu \geq 1$ number of parents, and one offspring. Parents generate an offspring and worst of them is eliminated according to their fitness values. Recombination is used firstly in this version of ESs, and population concept is introduced.

Modern versions of ESs are developed by H. P. Schwefel (1977). These are (μ+λ)-ES, and (μ,λ)-ES. They have μ number of parents and λ number of offspring. These two types of ESs are same in all manners except the selection property. In (μ+λ)-ES, μ number of individuals are selected from $\mu+\lambda$ number of individuals. On the other hand, μ number of individuals are selected from λ number of offspring in (μ,λ)-ES. In the latter one, living of the parents more than one generation is not permitted as similar to biological evolution concept.

Cai and Thierauf (1993) modifies ESs to solve optimization problems with discrete variables. Rudolph (1994) presents an adaptive reformulation for discrete optimization problems. Back and Schutz (1995) develops ESs by using a self-adaptive strategy parameter called as mutation probability. In the present study, (μ,λ)-ES is used and a modified version of Rudolph's approach mentioned above is used to handle discrete variables.

4.2 Definition

4.2.1 General

Evolution strategies technique is one of the global optimization methods, which employs an algorithm simulating the evolution theory of Darwin. In nature, those members of species that adopt to environment, will survive and those which cannot die. Fitness function replaces environment accommodation in evolution strategies. Fitness function in evolutionary strategies is conceptually identical to "adoption to environment" in nature. It is composed of two terms; objective function and constraints. **Objective function** defines purpose of the optimization. It can be minimization of weight or cost for a structural optimization problem. On the other hand, **constraints** are the limitations that solutions produced must satisfy. Allowable stress for members or deflection limits for nodes are examples of the possible constraints in structural problems. The purpose of the evolution process is to obtain ideal individual, which gives minimum or maximum result in objective function without violating any constraints.

Similar to biological evolution, evolution strategies also need a population to initiate evolution process. This population includes a certain number of individuals. Individual means a solution of the structural problem, which has variables such as size of members, location of nodes, etc. An initial set of individuals has to be generated before evolution process is started. These individuals may or may not violate some of the constraints. While the number of individuals violating some of the constraints is higher in first generations, it decreases with increasing number of generations.

Evolution for structural optimization is based on three operators named as recombination, mutation, and selection. **Recombination** creates a certain number of new offspring population from parent population (Hasançebi, 2007b). It is realized by interchanging values assigned on design variables between individuals for discrete variables, and by taking average of them for continuous variables. Although recombination is the main operator in some of other evolution algorithms, it is a secondary operator in ESs. Another and most important operator of ESs is mutation. **Mutation** is changing of the values assigned on the variables of an individual independent of other individuals. Strategy parameters, which define probability and intensity of mutation, are used to realize mutation process. Strategy parameters are also mutated according to situation of optimization process. This process is called as self-adaptation of strategy parameters in literature. Mutation is a main operator in ESs. Another operator used in optimization process is **selection**, which is similar to natural selection in biology and mimicks the concept of survival of the fittest in Darwin's theory. In evolution strategies, selection is a deterministic operator, and a predefined number of best individuals survive to form new generations.

Optimization process continues as a loop, until it converges to a solution or the number of generations is reached a predefined maximum generation number.

4.2.2 Constraint Handling

Structural optimization problems have certain and clearly defined constraints. However, evolutionary algorithms are unconstrained optimization methods, and thus they cannot directly handle constraints. One way to deal with constraints with ESs is to use **death penalty approach**. In this method, if an individual violates any constraint, it is eliminated. Nevertheless, This approach has certain shortcomings. If a local optimum is surrounded by constraints, it is difficult to avoid local optimum, since individuals are removed even in minor violent. In addition, if global optimum is surrounded with constraints, it is very hard to reach global optimum at this time (Ulusoy, 2002). For these reasons, death penalty approach prevents to search all design space exactly and efficiently. Therefore, it may be ineffective to use death penalty approach in evolution algorithms.

Another indirect way of handling with constraints is to modify objective function by an additional function, which is called as **penalty function**. Penalty function can be defined as a convertor, which converts the severity of constraint violation into additional weight. By this way, if an individual violate a constraint, the weight of the individual is increased as much as intensity of the violation. Therefore, in selection, chance of individuals to survive decreases with higher intensity of constraint violation. The term **fitness function** is used to define the sum of objective function and penalty function as seen in equation 4.1.

$$\mathrm{F(x)} = W(x) + P(x) \qquad (4.1)$$

F(x): Fitness function
W(x): Objective function
P(x): Penalty function

In this study, penalty function approach is used to deal with constraints. See Section 5.6 for details.

4.2.3 Recombination

Transfer of some parts of genetic material and joining to another one is defined as recombination in biology. It also has the similar function in evolution strategies. It provides the mixture of values assigned on the variables between two or more individuals. By using recombination, λ number of individuals is created from μ number of parents. Recombination is not used for only design variables, but also for strategy parameters. Recombination between two individual is named as sexual form. Recombination of one individual with all of other individuals is named as panmictic form. There are four different recombination types given by Back (1996) as follows,

i-) sexual discrete
ii-) panmictic discrete
iii-) sexual intermediate
iv-) panmictic intermediate

In type-i, variables of offspring take values chosen from two randomly selected individuals with equal probability. In type-ii, one individual is selected randomly, and variables of offspring take values chosen from this individual or other remaining individuals for each variable, separately and with equal probability. Type-iii is same with type-i, but this time arithmetic means of the values of two randomly selected individuals are taken by the offspring. In addition, type-iv is same with type-ii, but arithmetic means are taken by the offspring instead of probabilistic selection.

4.2.4 Mutation

Biological mutation is the sudden change in genetic material of the species. It is a complex behavior and generally harmful. Mutation concept in ESs is simpler and more beneficial compared to biological one. Mutation is the main operator in evolution strategies. It takes the most important role to search design space in optimization process. It is implemented in a randomized manner by using strategy parameters.

Strategy parameters constitute individuals by coupling with design variables for a successfully implemented optimization process. They orientated mutation process by arranging quantity and probability of mutation. Using constant strategy parameters is not sufficient, they also have to be modified according to situation of the population, this is called as "self-adaptation of strategy parameters". In other words, strategy parameters are also mutated. Whether variables are discrete or continuous has to be considered in selecting type of strategy parameters.

Optimization problems may have discrete or continuous variables. For example, in structural optimization, optimization of member sections in a structure is called as size optimization. Since in many cases there is only a set of sections available in the market, the variables assigned on member sizes are discrete variables. Changing the location of the nodes, by which members are connected to each other, is called as shape optimization. In this case, variables are continuous in a predefined range. Mutation of discrete and continuous variables are logically same, however completely different in implementation. Mutating discrete variables as continuous ones and then rounding them to closest discrete values is possible, nevertheless this method has some shortcomings. Firstly, the result may not give the optimum solution, and secondly, it may violate some of the constraints (Ulusoy, 2002).

Therefore several methods for mutation of discrete variables have been developed. Some of them are available in Cai et al. (1993), Rudolph (1994), and Back et al. (1995). A reformulation of Rudolph's approach developed by Hasançebi (2007a) is used in the mutation of discrete variables in this study.

4.2.4.1 Mutation of continuous variables

Mutation of continuous variables is given in Hasançebi (2007b) as follows,

$$c' = c + N(0, \sigma') \qquad (4.2)$$

c: continuous design variable
c': mutated continuous design variable
N(0,σ'): a normally distributed random number with mean, 0 and standard deviation, σ'

Strategy parameter used in equation (4.2) is actually a standard deviation used to find a normally distributed random number. This number, N(0,σ') determines the amount of variation in design variable. Before starting mutation of design variable, strategy parameter is required to be mutated first as follows,

$$\sigma' = \sigma + e^{N(0,\tau_c)} \qquad (4.3)$$

σ: previous strategy parameter
σ': mutated strategy parameter
$N(0,\tau_c)$: a normally distributed random number with mean, 0 and standard deviation, τ_c

In equation (4.3) τ_c is the learning rate and it is given by Back et al. (1995) as follows,

$$\tau_c = \frac{1}{\sqrt{n_c}} \qquad (4.4)$$

τ_c: learning rate for continuous design variables
n_c: total number of the continuous design variables

4.2.4.2 Mutation of discrete variables

Mutation of discrete variables is given in Kızılkan (2010) as follows,

$$d' = d + z \qquad (4.5)$$

d: discrete design variable
d': mutated discrete design variable

z is an integer number, which determines the amount of variation in mutated design variable. It is obtained by using following equation (4.6). r is generated anew for each design variable, z and its strategy parameter, ψ

$$z = \begin{cases} 0 & , if\ r > P_d' \\ g_1 - g_2 & , if\ r \leq P_d' \end{cases} \qquad (4.6)$$

z: an integer number used for discrete design variable
g_1,g_2: geometrically distributed random integer numbers

r is a uniformly distributed random number, which determines whether the variable mutate or not. P_d' is mutated strategy parameter or mutation probability changing between 0 and 1. As in mutation of continuous variables, strategy parameter is also mutated in the optimization of discrete variables. It is mutated as follows,

$$P_d' = \left(1 + \frac{1 - P_d}{P_d} \times e^{\gamma_d \times N(0,1)}\right)^{-1} \tag{4.7}$$

P_d: strategy parameter for discrete variables

$P_{d'}$: mutated strategy parameter for discrete variables

N(0,1): a normally distributed random number with mean, 0 and standard deviation, 1

γ_d is the learning rate for strategy parameter of discrete variables, which is recommended by Back et al. (1995) as follows,

$$\gamma_d = \frac{1}{\sqrt{2\sqrt{n_d}}} \tag{4.8}$$

γ_d: learning rate for strategy parameter of discrete variables

n_d: total number of discrete design variables

g_1 and g_2 are two geometrically distributed random integer numbers with expectation ψ. They are formulized as seen in equation (4.9). ψ is another strategy parameter, which is always kept over one. Like all other strategy parameters, it is also mutated as given in equation (4.10).

$$g_{1,2} = \left\lfloor \frac{\log(1 - r)}{\log(1 - \frac{1}{1 + \psi'})} \right\rfloor \tag{4.9}$$

g_1,g_2: geometrically distributed random integer numbers

r: uniformly distributed random integer numbers

$$\psi' = \begin{cases} \psi & , if\ r > P_d' \\ \psi \times\ e^{\tau \times N(0,1)} \geq 1.0 & , if\ r \leq P_d' \end{cases} \tag{4.10}$$

ψ: geometric distribution parameter for discrete variables

ψ': mutated geometric distribution parameter for discrete variables

r: uniformly distributed random integer numbers

P_d': mutated strategy parameter for discrete variables

τ is the learning rate for strategy parameter ψ, and it is given by Back et al. (1995) as follow,

$$\tau_d = \frac{1}{\sqrt{n_d}} \tag{4.11}$$

τ_d: learning rate for strategy parameter ψ

n_d: total number of the discrete variables

4.2.5 Selection

Selection is a deterministic operator in evolution strategies. As defined before, there are two selection methods in modern implementations of evolution strategies. They are (μ+λ)-ES, and (μ,λ)-ES. In (μ+λ)-ES, μ number of individuals are selected from μ+λ number of individuals, and in (μ,λ)-ES, μ number of individuals are selected from λ individuals. In other words, while parents may survive forever in (μ+λ)-ES, their life is only one generation in (μ,λ)-ES. At first sight, surviving of best individuals forever may be thought as a good thing. However, it makes difficult to leave local optima and maladapted strategy parameters. For these reasons, (μ,λ)-ES is used in this study.

Another issue for selection operator is offspring over parents ratio, λ/μ. It is important to provide an efficient evolution. If this ratio is too small, number of generations will increase to reach optimum. On the other hand, if the ratio is too large, it will increase time spent for each generation. Therefore, an optimum λ/μ ratio has to be selected. Ulusoy (2002) compares converging times of different λ/μ ratios for an 18-bar truss example and shows that a ratio changing between 5 and 7 performs quite well. In this study, using 5 is decided after some trials.

4.2.6 Termination

Termination is the stopping of algorithm when a termination criterion is satisfied. This criterion may be a previously defined maximum number of generations, or a limit of time, or whether a sufficient convergence is satisfied. In this study, a maximum number of generations, N_{gen} is defined and it is taken as equal to 2000.

4.2.7 Algorithm of Optimization Process

Algorithm used to computerize evolution strategies is given in Figure-9. In Step-1, counter of generation, t is set to 0 and population of zeroth generation is initialized that μ number of individual is created. Then, this population is evaluated for fitness in Step-2. According to obtained data, zero-th generation is recombined and mutated respectively to form λ number of offspring in Step-3. These offspring are evaluated according to fitness function in Step-4. After that, by using selection operator, μ number of parent is selected from λ number of offspring to form generation-(t+1) in Step-5. Generation counter, t is increased by one in Step-6. An if-condition is assigned to Step-7 to check whether the termination criterion is satisfied. If it is true, algorithm continues with Step-8, and it is terminated. In other case, it returns to Step-3.

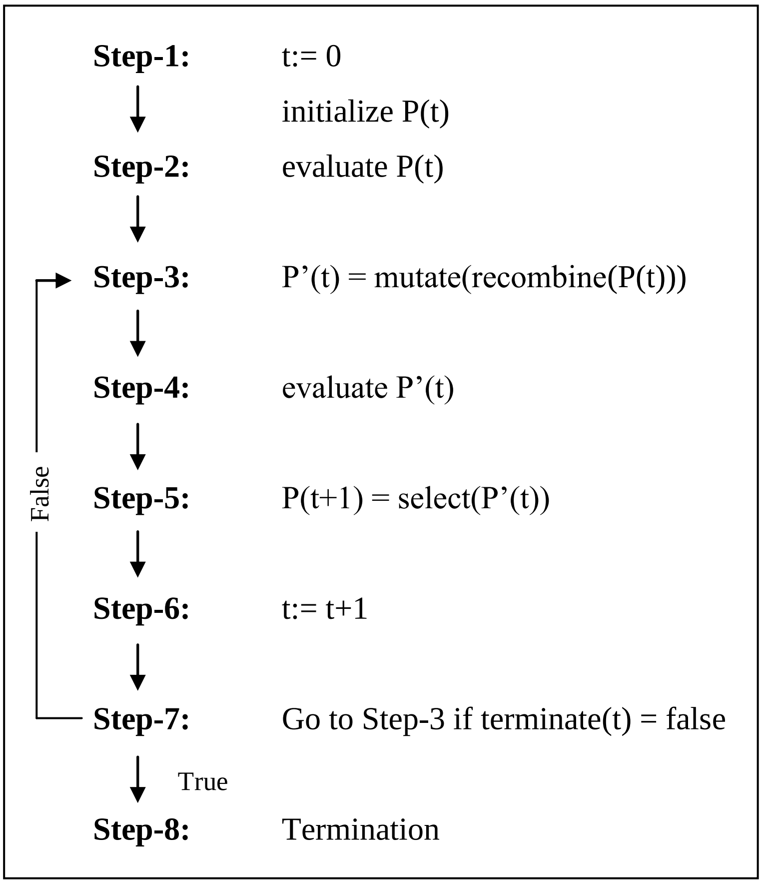

Figure – 9: Flowchart for algorithm of evolution strategies

CHAPTER 5

SIZE, SUPPORT and ELEVATION OPTIMIZATION of DOUBLE-LAYER GRIDS

5.1 General

Two-dimensional roof trusses are structures having only axially loaded members and have a simple design procedure. In conventional truss designs, three different sections are assigned for top chord, bottom chord, and web members respectively. Sometimes the number of the different sections assigned for web members can be more than one. However, total number of different size groups does not exceed four, or five. For this reason, members are designed according to members having maximum stresses in that group. This results many members to have sections larger than statically required. Nevertheless, grouping members to have same sizes has advantages in manufacturing and assembling stages. Therefore, weight optimization for conventional two-dimensional trusses is not a popular implementation in real-life practice. This situation is different in space trusses.

Space trusses are three-dimensional structures, which have similar structural behaviour with trusses. However, there are major differences in application. The most important difference is in connection details. There are many different connection types used in space trusses. The most common one, which is also the most popular one in Turkey, is spherical connections (See Part 3.2.6 for details). This connection type make assembling very easy, and make grouping members to have same size section needless. Every member of the space truss can take different section sizes and this increase the importance of optimization. Section size is not the only subject for optimization. In this study, elevation of the structure and horizontal restraints of the supports are also optimized beside the size optimization. Three different optimization combinations are used in this study. They are size optimization, simultaneous size and support optimization, and simultaneous size, support and elevation optimization. Formulations of ESs for these optimization combinations are presented in the following section.

5.2 Individual Representation

First, individuals are given separately for size, elevation and support optimization respectively. Then, individuals are represented for different combinations of optimization types.

5.2.1 Representation of size variables

One of the most common types of conventional structural optimization is size optimization. The optimum is found by identifying most appropriate sections for the members while keeping other design parameters constant. Although in literature there are many studies, which use continuous variables for member sections, it is generally not applicable in real life. The use of discrete sections is more realistic in which design variables can take values from a given set of sections available in the market. In this study, size variables are defined as discrete, and elements of discrete set are taken from steel sections used in the original, non-optimized design. The number of size variables is equal to number of members in the structure. In large structures having many members, high number of variables result in excessive computational effort. To handle this, member grouping can be applied as an alternative, which decrease the number of variables. It is applied by grouping members, which are predicted to have same section sizes. However, this generally reduces the efficiency of the optimization. Therefore, in this study, members are not grouped.

An individual that incorporates size variables can be formulated as follows,

$$d_I = (I, P_I, \psi) \tag{5.1}$$

In equation (5.1), I = $\{I_1,..,I_i,..,I_{nI}\}$ represents the size design variables of the structure. There is n_I number of independent variables and n_I is equal to total number of the members in the structure. P_I represents the set of strategy parameters used for size variables. It is also called as mutation probability. There are n_{P_I} numbers of mutation probabilities changing between 1 and n_I. Each size design variable is coupled with a mutation probability. Number of these mutation probabilities may vary between 1 and n_I. In case of $n_{P_I} < n_I$, first $n_{P_I} - 1$ size design variables are coupled with $\{P_{I1},\ldots,P_{Inp_I-1}\}$, one by one, and remaining design variables are coupled with P_{Inp_I}. In this study, n_{P_I} is taken as 1, which means that all size design variables coupled with same strategy parameter. Another strategy parameter, which is also called as geometric distribution parameter, ψ has the same number of parameters with P_I. It is also equal to 1 in present study. Coupling procedure is same with the previous strategy parameter.

5.2.2 Representation of support variables

Optimization of supports is conceptually similar to topology optimization. In topology optimization, an optimimum topology of a structure is sought by removing or restoring members or nodes in the structure. Similarly in support optimization, the restraint conditions at supports are optimized in two horizontal directions (x and y). Hence for each support two design variables are associated in this study. A support variable can take two values, 0, and 1. If it is equal to 1, then it is restrained, if it is 0, then it is not restrained. Number of support variables is equal to two times of that number of supports. There is a shortcoming about stability of the structure in some cases of restraints. In this situation, an unstable individual is removed from the population by assigning a very high penalty to it.

An individual incorparating support variables can be formulated as follows,

$$I_S = (s, P_s) \tag{5.2}$$

In equation (5.2), s = $\{s_1,..,s_i,..,s_{ns}\}$ represents the support design variables of the structure. There are n_s number of independent variables and n_s is equal to two times of number of the supports in the structure. P_s represents the strategy parameter for support design variables. It is also called as mutation probability since it determines the the probability of mutation operator. In this study, only one strategy parameter is used, which means that all size design variables are coupled with the same strategy parameter.

5.2.3 Representation of elevation variable

Elevation optimization is a typical of classical shape optimization defined in the literature. In shape optimization, best design is searched by changing locations of the nodes within a predefined range in any direction. In this study, all nodes at top layers are grouped into a single variable, which is allowed to change only in (z)-direction. Hence, a single variable used for changing the overall height of the double-layer grids system. This design variable is referred to elevation variable, which is used as a continuous variable. An individual incorporating the elevation variable can be formulated as follows,

$$I_h = (h, \sigma) \tag{5.3}$$

In equation (5.3), h represents the elevation design variable of the structure. σ represents the strategy parameter used for the elevation variable. It is used as standard deviation in mutation of elevation design variable.

5.2.4 Combined representation of variables

In the study, size optimization is implemented with and without elevation and support optimization. Hence, three different models of optimization are studied. These are size, size + support, and size + support + elevation optimizations. Their individual representations are as follows.

<u>Optimization</u>	<u>Individual</u>
Size	$d = (I, P_I, \psi)$
Size + Support	$d = (I, s, P_I, P_s, \psi)$
Size + Support + Elevation	$d = (I, h, s, P_I, \psi, P_s, \sigma)$

I: Size design variables

h: Elevation design variable

s: Support design variables

P_I: Strategy parameter for size design variables

ψ: Geometric distribution parameter for size design variables

P_s: Strategy parameter for support design variables

σ: Strategy parameter for elevation design variable

5.3 Recombination

In part 4.2.3, definition and types of recombination operator are given in details. Here, only recombination types used for different types of design variables and strategy parameters are given. Panmictic discrete recombination, (type-ii) is used for size and support design variables. Strategy parameters of them are recombined by sexual intermediate recombination (type-iii). Finally, elevation variable and its strategy parameter are again recombined by sexual intermediate recombination (type-iii).

5.4 Mutation

Mutations of three different design variables, size, support and elevation, and their strategy parameters are defined in the following sections.

5.4.1 Mutation of size variables

Size variables are discrete variables. Mutation procedure of discrete variables is given in part 4.2.4.2 in details. Here, it is rearranged for size variables. Mutation of size variables is given as follows,

$$I_i' = I_i + z_i \qquad (5.4)$$

I_i: i-th size design variable
I_i': mutated i-th size design variable

z_i is an integer number assigned for i-th variable, which determines the amount of variation in mutated design variable. It is obtained by using equation (5.5).

$$z_i = \begin{cases} 0 & , if\ r_i > P_I' \\ g_{i,1} - g_{i,2} & , if\ r_i \leq P_I' \end{cases} \qquad (5.5)$$

z_i: an integer number assigned for i-th size design variable
$g_{i,1}$,$g_{i,2}$: geometrically distributed random integer numbers for i-th size design variable
r_i: uniformly distributed random integer numbers for i-th size design variable

P_I' is mutated strategy parameter or mutation probability changing between 0 and 1. It is mutated as follows,

$$P_I' = \left(1 + \frac{1 - P_I}{P_I} \times e^{\gamma_I \times N(0,1)}\right)^{-1} \qquad (5.6)$$

P_I: strategy parameter for size design variables

P_I': mutated strategy parameter for size design variables

N(0,1): a normally distributed random number with mean, 0 and standard deviation, 1

γ_I is the learning rate for strategy parameter of size variables, which is given as follows,

$$\gamma_I = \frac{1}{\sqrt{2\sqrt{n_I}}} \qquad (5.7)$$

γ_I: learning rate for strategy parameter of size variables

n_I: total number of size design variables

$g_{i,1}$ and $g_{i,2}$ are two geometrically distributed random integer numbers with expectation ψ. They are given in equation (5.8). ψ is another strategy parameter, which is also mutated as given in equation (5.9).

$$g_{i,1}, g_{i,2} = \left\lfloor \frac{\log(1 - r_i)}{\log(1 - \frac{1}{1 + \psi'})} \right\rfloor \qquad (5.8)$$

r_i: uniformly distributed random integer numbers for i-th size design variable

$$\psi' = \begin{cases} \psi & , if\ r_i > P_I' \\ \psi \times\ e^{\tau_I \times N(0,1)} \geq 1 & , if\ r_i \leq P_I' \end{cases} \qquad (5.9)$$

ψ: geometric distribution parameter for size design variables

ψ': mutated geometric distribution parameter for size design variables

P_I': mutated strategy parameter for size design variables

τ_I is the learning rate for strategy parameter ψ, and it is taken as follow in this study,

$$\tau_I = \frac{1}{\sqrt{n_I}} \qquad (5.10)$$

τ_I: learning rate for geometric distribution parameter, ψ
n_I: total number of size design variables

5.4.2 Mutation of elevation variable

Elevation variable is the only continuous variable in this study. Mutation procedure of continuous variables is given in part 4.2.4.1 in details. Here, it is rearranged for elevation variable. Mutation of elevation variable is given as follows,

$$h' = h + N(0, \sigma') \qquad (5.11)$$

h: elevation design variable
h': mutated elevation design variable
N(0,σ'): a normally distributed random number with mean, 0 and standard deviation, σ'

Strategy parameter, σ is mutated first as follows,

$$\sigma' = \sigma + e^{N(0,\tau_h)} \qquad (5.12)$$

σ: previous strategy parameter for elevation design variable
σ': mutated strategy parameter for elevation design variable

$N(0,\tau_h)$: a normally distributed random number with mean, 0 and standard deviation, τ_h

τ_h is the learning rate for elevation design variable and it is found as 1 by using equation (4.4)

5.4.3 Mutation of support variables

Support variables are discrete variables. However, in support optimization, a variable can take only two values, 0 or 1. Therefore, mutation of support variables is simpler than given for other discrete variables. Here, it is modified for support variables. Second strategy parameter is unnecessary in this case, so it is removed and a switch operator is introduced as given in equation (5.13).

$$s_j' = \begin{cases} switch(s_j) & if\ r_j \leq P_s' \\ s_j & if\ r_j > P_s' \end{cases} \tag{5.13}$$

s_j: j-th support design variable
s_j': mutated j-th support design variable
r_j: uniformly distributed random integer numbers for j-th support design variable

Switch operator changes value of s_j from zero to one, or from one to zero. Ps' is mutated strategy parameter or mutation probability changing between 0 and 1. It is mutated as follows,

$$P_s' = \left(1 + \frac{1 - P_s}{P_s} \times e^{\gamma_s \times N(0,1)}\right)^{-1} \tag{5.14}$$

P_s: previous strategy parameter for support design variables
P_s': mutated strategy parameter for support design variables

N(0,1): a normally distributed random number with mean, 0 and standard deviation, 1

γ_s is the learning rate for strategy parameter of support variables, which is given as follows,

$$\gamma_s = \frac{1}{\sqrt{2\sqrt{n_s}}} \tag{5.15}$$

γ_s: learning rate for strategy parameter of support variables
n_s: total number of support design variables

5.5 Initial Population

Optimization process starts with μ number of individuals. Every individual has design variables and strategy parameters with their initial values. In a conventional ESs, initial values of design variables are randomly chosen from a previously defined set of sections or limit of range. Unlike conventional implementation**,** an **iterative design method** is used to obtain an initial set of individuals in this study. In this method, the smallest section is initially assigned to all members and these are increased if calculated stresses are higher than the allowable stresses. In size optimization, support conditions and height of the structure are kept constant, and so only one initial design is obtained. This makes all initial population same for size optimization. In size and support optimization, only height of the structure is kept fixed and, iterative stress-based design gives many solutions with different support conditions. Best μ number of them is selected as initial population of ESs. Similarly, in simultaneous size, support and elevation optimization, variety of solutions are found with different combinations of elevation and support conditions, and best μ number of them is selected as initial population.

The idea for implementing the iterative stress-based design technique is to enforce the algorithm from a good point or a set of good points in the design space. It is found that when the optimization is initiated from randomly generated initial solutions, it takes quite long time for the algorithm to reach a good design point due to a very high number of design variables.

In this study, λ/μ ratio is taken as 5, and number of parents is taken as 10. Initial values for strategy parameters are defined according to experiments. Mutation probabilities for size and support design variables, P_I, and P_s are set to 0.01 and 0.25 respectively.. Another strategy parameter for size design variables, ψ is set to 5 (Hasançebi, 2007b). Strategy parameter of elevation design variables, σ is defined by an equation as follows,

$$\sigma_i = 0.1 \times h_i \tag{5.16}$$

h_i: initial height of the structure

5.6 Fitness Evaluation

The main concept in evolution strategies is to provide “survival of the fittest”. To realize this, a fitness function has to be defined and selection operator is used to eliminate insufficient individuals according to this function. Fitness function has to consider two requirements; objective and constraints of optimization. In structural optimization problem, objective is generally weight, or cost minimization. In this study, objective is minimization of the weight, which is formulated as follows,

$$Objective\ function, W = \rho \sum_{i=1}^{n_I} (A_i \times L_i) \tag{5.17}$$

ρ: unit weight of structural material

A_i: sectional area of i-th member

L_i: length of i-th member

n_I: total number of size design variables

The ways to handle constraints in evolution strategies is discussed in section 4.2.2 and a penalty function is formulized to prevent constraint violation as follows,

$$Penalty\ function, P = W \times \alpha \times \sum_{k=1}^{n_{cn}} (g_k) \qquad (5.18)$$

W: objective function or weight of the structure

α: penalty coefficient

n_{cn}: total number of the constraints

g_k: k-th normalized constraint

α is the parameter designating intensity of penalty. It can be a static or constant value. However, it is adaptively implemented in this study. In other words, it is taken as a dynamic coefficient, which changes according to feasibility of the best design in previous population. Optimum results are generally located in constraint boundaries and the logic behind penalty coefficient is to keep search along these boundaries. If an individual obey all of the constraints, it is named as feasible, else it is named as infeasible. Penalty coefficient is decreased if best design of previous generation is feasible, and it is increased in other case. Penalty coefficient is given in Hasançebi (2008) as follows,

$$\alpha(t) = \begin{cases} (1/f) \times \alpha(t-1) & if\ b(t-1)\ is\ feasible \\ f \times \alpha(t-1) & if\ b(t-1)\ is\ infeasible \end{cases} \qquad (5.19)$$

f: an arbitrary constant

α(t): penalty coefficient in generation(t)

α(t-1): penalty coefficient in generation(t-1)

b(t-1): best design in generation(t-1)

Arbitrary constant, f is taken as equal to 1.1. Normalization of k^{th} constraint, g_k in equation (5.18) is obtained as follows,

$$g_k = \max\left\{0, \frac{w_k}{w_{all}} - 1\right\} \quad (5.20)$$

w_k: available response for k-th constraint
w_{all}: allowable response for k-th constraint

Finally, fitness function is formulated as the total of objective and penalty functions as follows,

$$Fitness\ function, F = W \times \left(1 + \alpha \times \sum_{k=1}^{n_c} (g_k)\right) \quad (5.21)$$

5.7 Constraints

Constraints used for optimization of space trusses can be defined as allowable tension and compression stresses for members, and deflection limit for nodes. These constraints are exactly same with the ones used in real-life. Constraints functions used in this study are as follows,

$$\frac{\sigma_{t-i}}{\sigma_{t-all}} - 1 \leq 0 \quad (4.33)$$

$$\frac{\sigma_{c-i}}{\sigma_{c-all}} - 1 \leq 0 \quad (4.34)$$

$$\frac{L_s}{\Delta_{max}} - 300 \geq 0 \quad (4.35)$$

$$\lambda_i - 250 \leq 0 \quad (4.36)$$

σ_{t-i}: maximum tensile stress on i-th member

σ_{t-all}: allowable tensile stress

σ_{c-i}: maximum compressive stress on i-th member

σ_{c-all}: allowable compressive stress

Δ_{max}: maximum vertical deflection

L_s: span length for the node having maximum vertical deflection

λ_i: slenderness ratio of i-th member

Evaluations of allowable stresses for tensile and compressive forces are given in Chapter-3.

5.8 Optimization Software – OFES

OFES is a computer program developed by Dr. Oğuzhan Hasançebi for optimization of steel frames with evolution strategies (See Figure.-10). It is a design kit, which directly computerize the algorithm given in part 4.2.7. It needs structural analyzes in Step-2 and Step-4 of this algorithm. However, it does not have an interior analysis patch, instead it uses an exterior software package to get structural analysis results. Sap2000 v.7 evaluates all stresses on members, and deflections on the nodes and OFES uses them in fitness evaluation process.

In this study, OFES is developed for optimization of space trusses. Some additional outputs about manufacturing and assembling details of the structure are attached to design real structures. Extra input and output files are prepared for this purpose. These main and extra files are defined briefly to show how to use this kit.

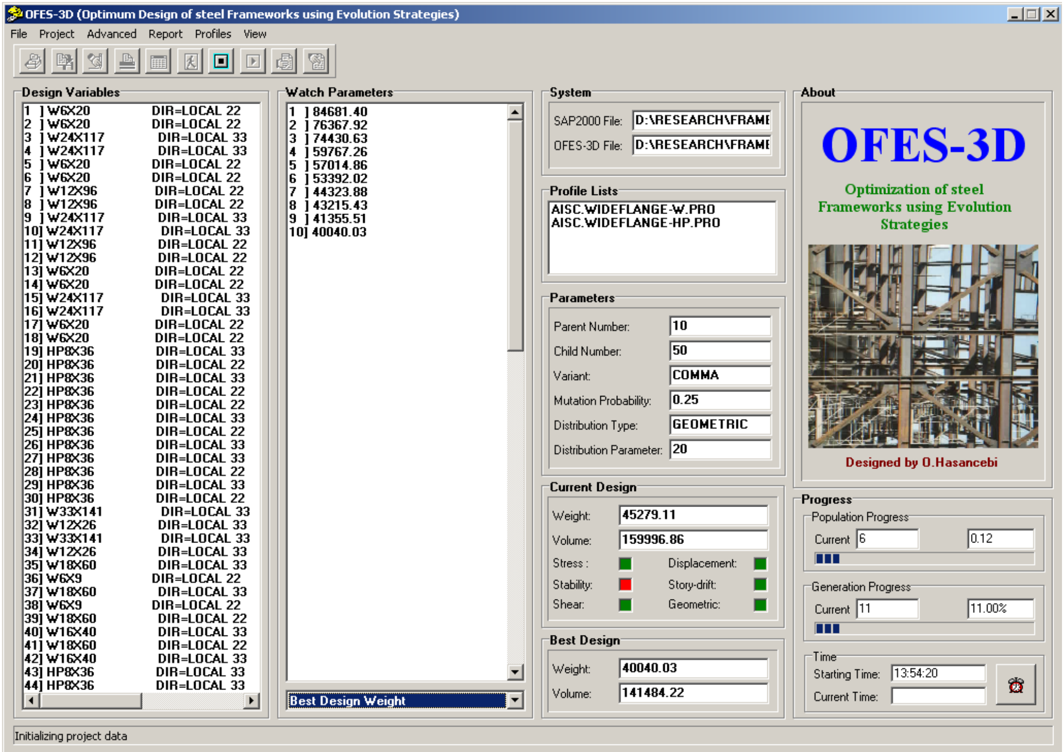

Figure –10: OFES – Main Menu

5.8.1 Input files:

IES File: It is one of the main input files for OFES. Some parameters for ESs and name of profile lists for members are introduced in this file. In addition, grouping of members and design types of members, beam, column, or truss, are also defined in this file. Many of these input data can also be changed later by using program interface.

S2k File: It is actually, a Sap2000 v.7 output file, which includes all geometric and connectivity properties of the structure, supports, loads, and combinations used in the design. S2k file is a text file, which is easy to use in other programs. Before starting to use OFES, structural model is prepared in Sap2000, all of the loads, combinations, and supports are defined. Finally, this s2k file is created as a text file, and given to OFES as an input file.

PRO File: Profile list and properties of sections in this list are taken from this file.

SPH File: It defines available sphere diameters to be used in detailing. This file is not required for optimization process.

CON File: It defines connection types to be used in detailing. Dimensions of connection members like bolts, nuts and conics are dependent each other, so some certain connection types are defined by listing different combinations of these connection members. Bolt, nut, and conic dimensions for each type are included in this file. It is also not required for optimization process.

WEI File: It defines the weights of the available conics. Weight of other connection elements can be computed by program. However, conics are not standard and so their weights are taken as input.

MIN File: It defines minimum dimensions of connection elements like, nuts, bolts, and conics for each member sizes.

5.8.2 Output Files:

DBES File: It is actually a complete design report file including geometric and connectivity properties of the structure. Loads, combinations, and boundary conditions are included. Stresses on members, deflection of nodes are given in details. All required information for manufacturing and assembling of the structure is also given. Especially, sphere types, which are determined according to angle and diameters of holes on the sphere, are showed clearly.

S2k File: Program gives also an s2k file to open best design found by OFES in Sap2000 v.7. It provides to inspect last situation of the structure in Sap2000.

CHAPTER 6

NUMERICAL EXAMPLES

6.1 General

In this chapter, the optimum design studies are carried out on six different double-layer grids that are taken from real-life industrial applications and that are previously designed by Polarkon Steel Structures Company using the FrameCAD software automated for conventional design of such systems. Number of the members of space trusses in the examples changes between 792 and 4412. Loads, combinations and profile lists of the members are taken as exactly same as original ones to make a fair comparison between the optimum and conventional solutions. Both original and optimized structures are designed according to Turkish specification. First, only size optimization is implemented, then size and support optimizations are implemented simultaneously. Finally, all size, support, and elevation optimizations are used at the same time. Results obtained by all these optimization processes, comparison of them with each other and non-optimum ones are given in the following parts of this chapter.

6.1.1 Method of Structural Analysis

Structural analyses of the structures are performed by using Sap2000 v.7, which is popular and reliable structural analysis software. Sap2000 analyze structural models by using finite element method. Geometric and connectivity properties, supports, loads, and load combinations are all included in these models. If there are any columns, on which the space trusses are supported, they are also defined in structural models. After analyzing process, forces acting on each member are given to OFES as an input file for optimization process.

6.1.2 Assumptions

Structural model is simplified by making some basic assumptions. Most important one is that spherical nodes do not transfer any moment. Therefore, members cannot take any moment or shear. Members are designed for only axially tension and compression forces. Moreover, modulus of elasticity, E, and thermal expansion coefficient, C are taken as equal to $2100 t/cm^2$ and 0.000012/C, respectively as given in Turkish specification, TS648. In addition, effective length of members is conservatively taken from centre to centre of the spheres for buckling control of the members.

6.1.3 Specifications

Turkish Building Code for Steel Structures, TS648 is used in the design of pipe members. Other members like bolts and nuts are designed according to allowable stresses used by Polarkon SSC. Design requirements and allowable capacities for each part of double-layer grids are explained in Chapter-3. Design Load for Buildings, TS498 is considered to determine snow and wind loads in the original designs. Amounts of these loads are changed in some examples as parallel to original designs. Earthquake load is evaluated as equivalent static load according to 2007 Turkish Earthquake Specification and assigned as point loads on every node in the structural model. All requirements are taken exactly same with original non-optimum designs.

6.2 Examples

6.2.1 792-bar Space Truss

6.2.1.1 General Properties

Location	:	Northern Cyprus / Eastern Mediterranean University
Main Dimensions	:	34.0m x 26.1m
Area	:	887m^2
Critical span	:	26.1m
Module size	:	2.90m x 3.09m
Module height	:	2.25m
Number of members	:	792
Number of nodes	:	219
Number of supports	:	8
Column sections	:	supported on slab (rigid in horizontal directions)
Column lengths	:	-

6.2.1.2 Loads

L1: Dead Load	:	Own weight
L2: Purlins and Claddings Load	:	20 kg/m^2
L3: Service Load	:	30 kg/m^2
L4: Live/Snow Load	:	60 kg/m^2
L5: Wind Load (left to right)	:	100 kg/m^2
L6: Wind Load (right to left)	:	100 kg/m^2
L7: Wind Load (behind to front)	:	100 kg/m^2
L8: Wind Load (front to behind)	:	100 kg/m^2
L9: Wind Load (bottom to top)	:	0 kg/m^2
L10: Wind Load (top to bottom)	:	0 kg/m^2
L11: Earthquake (x-dir)	:	Region=I / R=5 / I=1.2 / S(T)=2.5
L12: Earthquake (y-dir)	:	Region=I / R=5 / I=1.2 / S(T)=2.5
L13: Temperature Difference	:	±30 C

6.2.1.3 Load Combinations

Load combinations used in 792-bar space truss designs for Turkish Specification are given in Table 6.1. They are exactly identical to those used when the conventional design of the system is obtained. First two combinations are accepted as EI combinations and remaining combinations are accepted EIY combinations. In design stage, this distinction is considered to determine allowable stresses. Combinations used in original, non-optimum design do not include any wind forces in upward direction. Actually, this is not a correct practice. However, to make an accurate comparison, these wind forces are not included either in combinations used in this study. Four different combinations are defined for wind loads according to blowing directions including left to right, right to left, front to behind and behind to front. The coefficients of wind load cases for different directions are the coefficients given in TS498 defined for pressure and suction surfaces (See Part 3.3.1.3 for details).

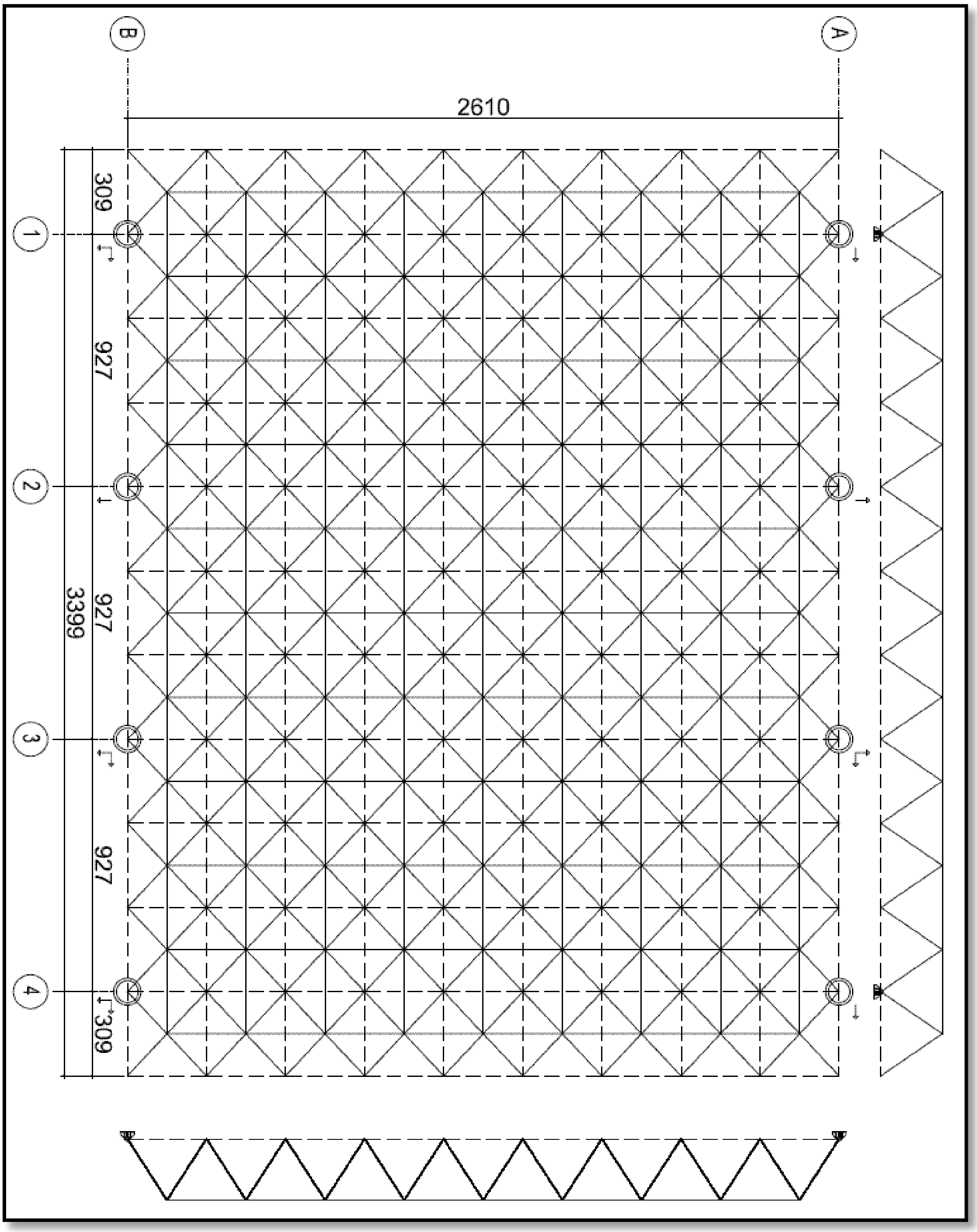

Figure –11: Plan View of 792-bar Space Truss with Original Supports

Table 6.1: Load Combinations for 792-bar Space Truss

Comb. No.	LOAD CASE												
	L1	**L2**	**L3**	**L4**	**L5**	**L6**	**L7**	**L8**	**L9**	**L10**	**L11**	**L12**	**L13**
1	1	1	1	0	0	0	0	0	0	0	0	0	0
2	1	1	1	1	0	0	0	0	0	0	0	0	-1
3	1	1	1	1	0,4	-0,2	-0,2	-0,2	0	0	0	0	-1
4	1	1	1	1	-0,2	0,4	-0,2	-0,2	0	0	0	0	-1
5	1	1	1	1	-0,2	-0,2	0,4	-0,2	0	0	0	0	-1
6	1	1	1	1	-0,2	-0,2	-0,2	0,4	0	0	0	0	-1
7	1	1	1	0,5	0,8	-0,4	-0,4	-0,4	0	0	0	0	1
8	1	1	1	0,5	-0,4	0,8	-0,4	-0,4	0	0	0	0	1
9	1	1	1	0,5	-0,4	-0,4	0,8	-0,4	0	0	0	0	1
10	1	1	1	0,5	-0,4	-0,4	-0,4	0,8	0	0	0	0	1
11	1	1	1	1	0	0	0	0	0	0	1	0	1
12	1	1	1	1	0	0	0	0	0	0	-1	0	-1
13	1	1	1	1	0	0	0	0	0	0	0	1	1
14	1	1	1	1	0	0	0	0	0	0	0	-1	-1

6.2.1.4 Profile List

Profile list of pipe sections used in 792-bar space truss is given in Table 6.2.

Table 6.2: Profile List Used In 792-bar Space Truss

Outer Diameter (mm)	Thickness (mm)	Grade
48.3	2.50	St37
48.3	3.00	St37
60.3	2.50	St37
60.3	3.00	St37
60.3	3.40	St37
76.1	3.40	St37
88.9	3.76	St37
114.3	4.05	St37
114.3	4.50	St37
139.7	4.50	St37
159.0	4.50	St37
219.1	4.50	St37
219.1	6.00	St37
219.1	6.00	St52
219.1	7.00	St52
219.1	11.00	St52

6.2.1.5 Results

In Table 6.3 , results obtained from size optimization, size and support optimization, and all size, shape and elevation optimization are given in details. The weights of various parts of the double-layer grid obtained in different case studies are also tabulated in this table.

Table 6.3: Weight and Displacement Ratios of 792-bar Space Truss before and after Optimization

		FrameCAD	OFES		
			Size	Size+Supp	Size+Supp+Elev
Weight (kg)	Pipe	10142	9702	9547	9499
	Bolt	275	230	229	233
	Nut	154	133	131	135
	Conic	506	471	453	470
	Sphere	833	423	420	424
	TOTAL	**11910**	**10959**	**10780**	**10761**
Displacement / Span Ratio		**1/534**	**1/539**	**1/515**	**1/463**

In Table 6.4, the percentages of weight reductions for each optimization type are presented. 8.0% of the weight can be reduced by using only size optimization in this example. When supports are also optimized with member sizes simultaneously, this percent reaches to 9.5%. Releases in restraints of optimized supports are shown in Figure-12. Arrow means support is released at those directions. Finally, 9.6% decrease is provided by implementation of all size, support, and elevation optimization at the same time. Height of structure changes from 2.25m to 2.07m in optimum design. This change provides only 0.1% more reduction in total weight of structure. It is also seen that spheres have the largest percent, 49.1% and pipe members have the smallest percent as equal to 6.3%. FrameCAD gives higher diameter of spheres than required by keeping a higher safety limit. However, smaller spheres can be obtained by decreasing or removing tolerances. In this study, spheres are calculated with zero tolerance, which is a common application in real-life.

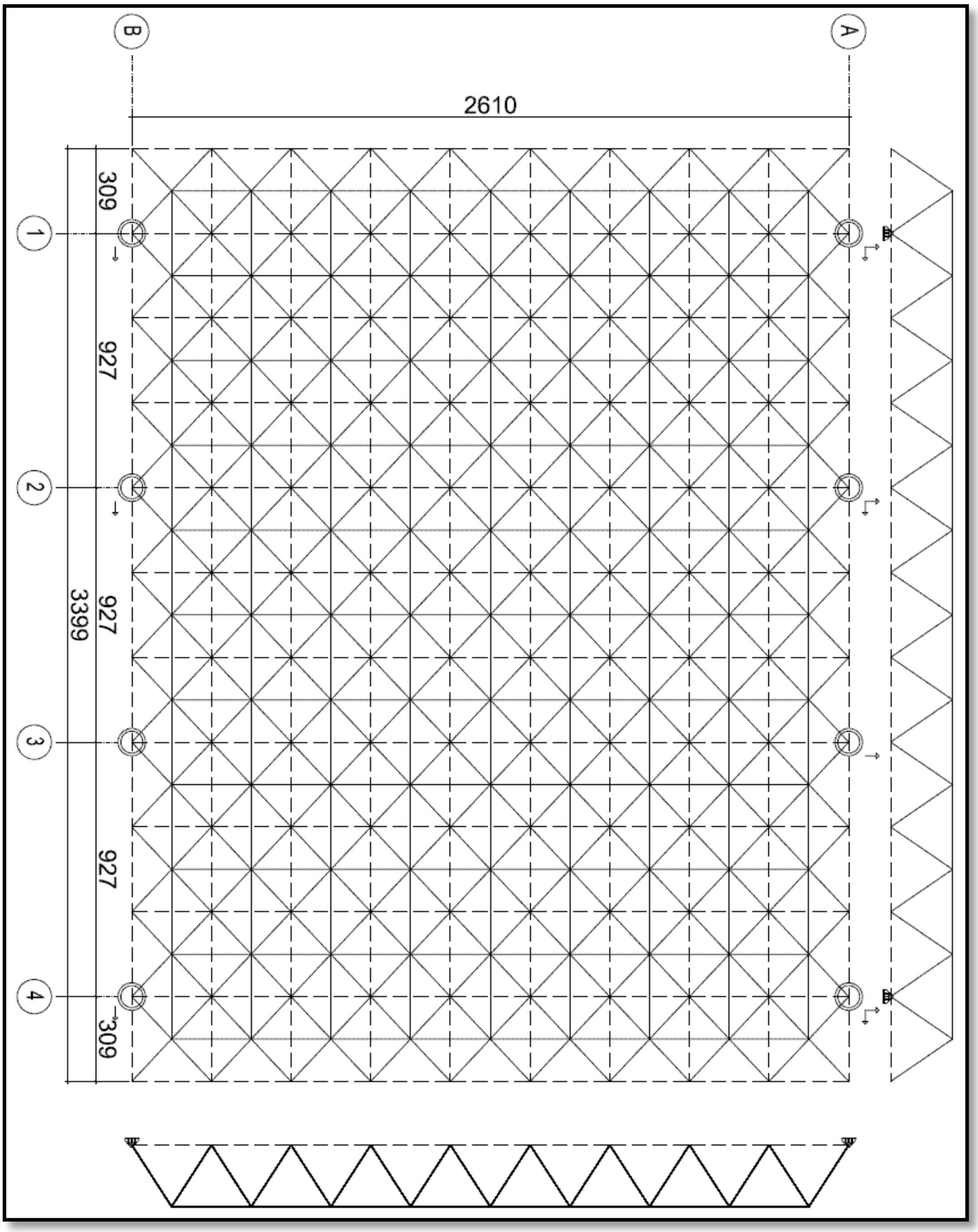

Figure –12: Plan View of 792-bar Space Truss with Optimized Supports

Table 6.4: Reduction Percents in Weight of 792-bar Space Truss after Optimization

	OFES		
	Size	**Size+Supp**	**Size+Supp+Elev**
Pipe	4.3	5.9	6.3
Bolt	16.4	16.7	15.3
Nut	13.6	14.9	12.3
Conic	6.9	10.5	7.1
Sphere	49.2	49.6	49.1
TOTAL	**8.0**	**9.5**	**9.6**

6.2.2 1360-bar space truss

6.2.2.1 General Properties

Location : İzmir

Main Dimensions : 48.5m x 31.3m

Area : 1514m^2

Critical span : 31.3m

Module size : 3.13m x 3.50m

Module height : 2.00m

Number of members : 1360

Number of nodes : 365

Number of supports : 22

Column sections : 55cm x 55cm (16 of 22)
40cm x 40cm (6 of 22)

Column lengths : 6.0m

6.2.2.2 Loads

L1: Dead Load	:	Own weight
L2: Purlins and Claddings Load	:	30 kg/m^2
L3: Service Load	:	20 kg/m^2
L4: Live/Snow Load	:	75 kg/m^2
L5: Wind Load (left to right)	:	80 kg/m^2
L6: Wind Load (right to left)	:	80 kg/m^2
L7: Wind Load (behind to front)	:	80 kg/m^2
L8: Wind Load (front to behind)	:	80 kg/m^2
L9: Wind Load (bottom to top)	:	80 kg/m^2
L10: Wind Load (top to bottom)	:	80 kg/m^2
L11: Earthquake (x-dir)	:	Region=I / R=3 / I=1.2 / S(T)=2.5
L12: Earthquake (y-dir)	:	Region=I / R=3 / I=1.2 / S(T)=2.5
L13: Temperature Difference	:	±30 C

6.2.2.3 Load Combinations

Load combinations used in 1360-bar space truss designs for Turkish Specification are given in Table 6.5. Combinations used in original design include wind force in downward direction. However, wind never pushes the structure from top to bottom, therefore it is not correct. Nevertheless, to carry out an exact comparison, it is also applied in this study.

6.2.2.4 Profile List

Profile list of pipe sections used in 1360-bar space truss is given in Table 6.6.

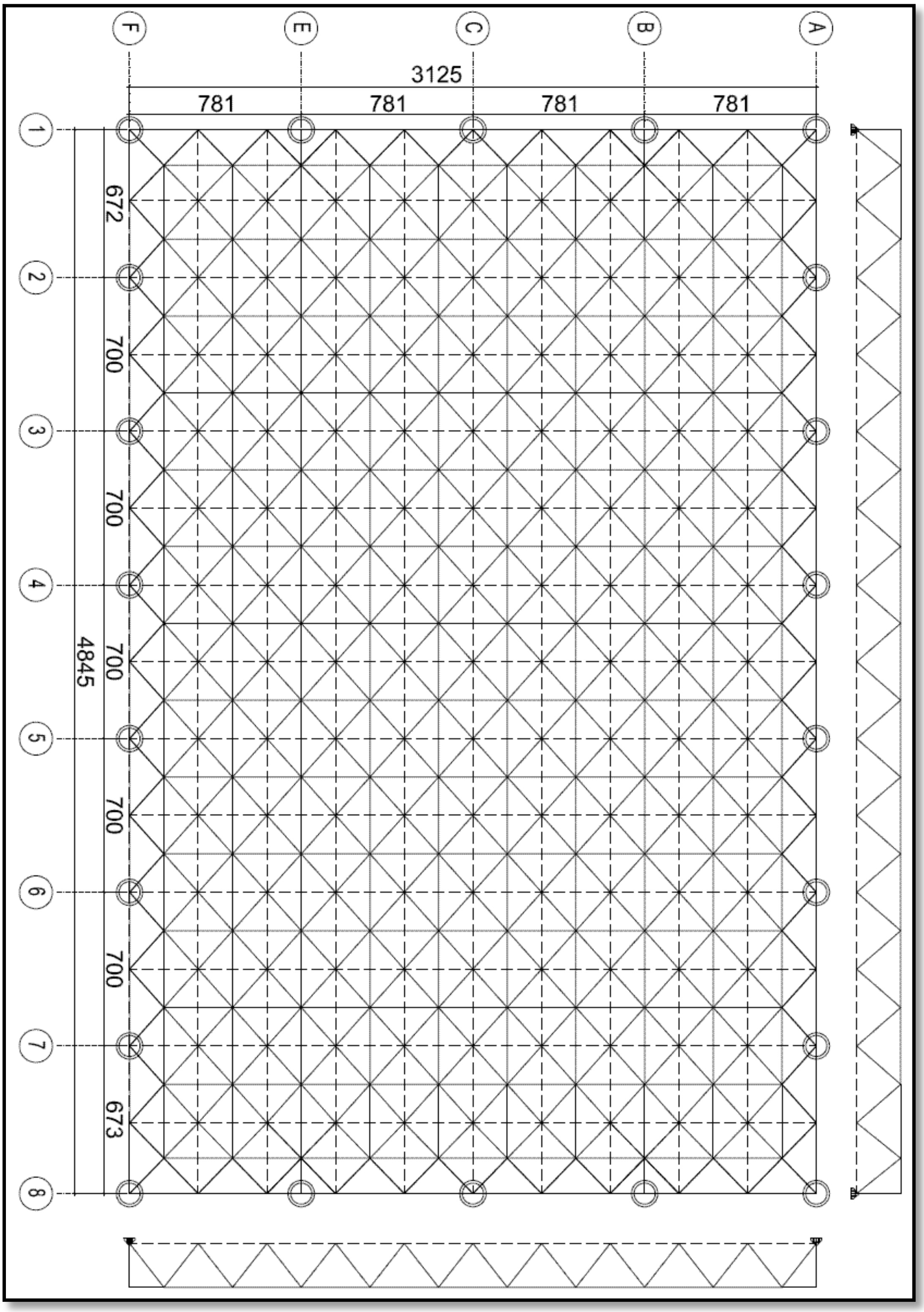

Figure –13: Plan View of 1360-bar Space Truss with Original Supports

Table 6.5: Load Combinations for 1360-bar space truss

Comb. No.	LOAD CASE												
	L1	L2	L3	L4	L5	L6	L7	L8	L9	L10	L11	L12	L13
1	1	1	1	0	0	0	0	0	0	0	0	0	0
2	1	1	1	1	0	0	0	0	0	0	0	0	0
3	1	1	1	1	0,8	-0,4	-0,4	-0,4	0	0	0	0	0
4	1	1	1	1	-0,4	0,8	-0,4	-0,4	0	0	0	0	0
5	1	1	1	1	-0,4	-0,4	0,8	-0,4	0	0	0	0	0
6	1	1	1	1	-0,4	-0,4	-0,4	0,8	0	0	0	0	0
7	1	1	1	0	0	0	0	0	0,8	0	0	0	0
8	1	1	1	1	0	0	0	0	0	0.8	0	0	0
9	1	1	1	1	0	0	0	0	0	0	1	0.3	0
10	1	1	1	1	0	0	0	0	0	0	1	-0.3	0
11	1	1	1	1	0	0	0	0	0	0	-1	0.3	0
12	1	1	1	1	0	0	0	0	0	0	-1	-0.3	0
13	1	1	1	1	0	0	0	0	0	0	0.3	1	0
14	1	1	1	1	0	0	0	0	0	0	-0.3	1	0
15	1	1	1	1	0	0	0	0	0	0	0.3	-1	0
16	1	1	1	1	0	0	0	0	0	0	-0.3	-1	0
17	1	1	1	1	0	0	0	0	0	0	0	0	-1
18	1	1	1	0	0	0	0	0	0	0	0	0	1

Table 6.6: Profile List Used in 1360-bar Space Truss

Outer Diameter (mm)	Thickness (mm)	Grade
48.3	3.00	St37
60.3	3.00	St37
60.3	3.40	St37
76.1	3.40	St37
88.9	3.76	St37
114.3	4.05	St37
114.3	4.50	St37
139.7	4.50	St37
159.0	4.50	St37
219.1	4.50	St37
219.1	6.00	St37
219.1	6.00	St52
219.1	7.00	St52
219.1	11.00	St52

6.2.2.5 Results

The results obtained by different optimization models are shown in Tables 6.7 and 6.8. In this example, Size optimization leads to a weight reduction as much as 22.7%. Simultaneous optimization of size and supports results in additional weight reduction of 1.5%, and hence the total weight reduction is accumulated to 24.2%. Figure-14 shows restraint details of the supports after optimization. Finally, by introducing elevation optimization with size and support optimization at the same time, the reduction rate reaches to 24.5% as compared to FrameCAD solution. Height of the structure is changed from 2.00m in the initial design to 2.08m after optimization.

Table 6.7: Weight and Displacement Ratios of 1360-bar Space Truss before and after Optimization

		FrameCAD	OFES		
			Size	Size+Supp	Size+Supp+Elev
Weight (kg)	Pipe	25981	21958	21497	21484
	Bolt	851	619	620	589
	Nut	567	437	444	419
	Conic	1700	1403	1370	1341
	Sphere	3991	1151	1153	1156
	TOTAL	**33090**	**25568**	**25084**	**24989**
Displacement / Span Ratio		**1/401**	**1/327**	**1/313**	**1/325**

Table 6.8: Reduction Percents in Weight of 1360-bar Space Truss after Optimization

	OFES		
	Size	Size+Supp	Size+Supp+Elev
Pipe	15.5	17.3	17.3
Bolt	27.3	27.1	30.8
Nut	22.9	21.7	26.1
Conic	17.5	19.4	21.1
Sphere	71.2	71.1	71.0
TOTAL	**22.7**	**24.2**	**24.5**

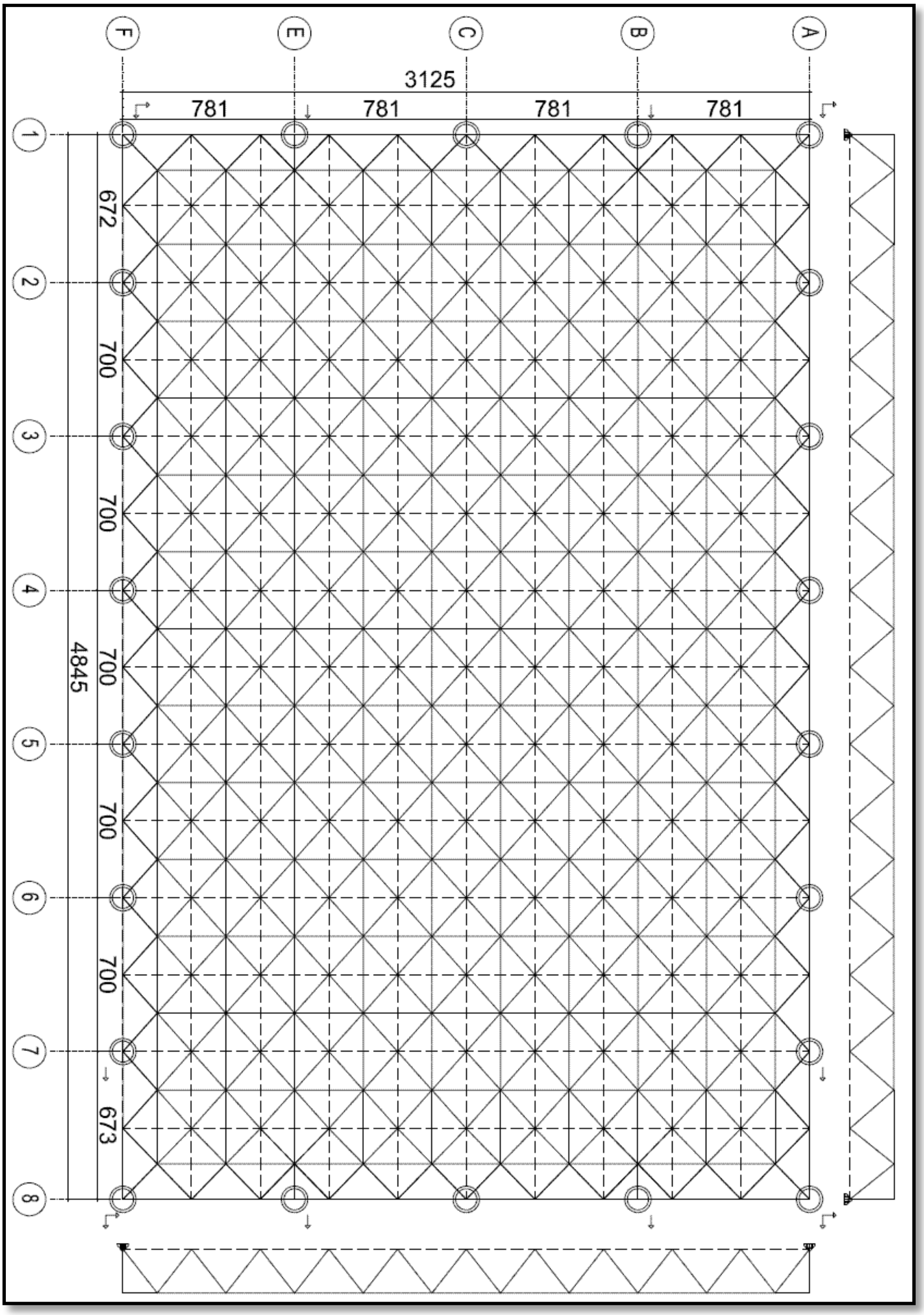

Figure –14: Plan View of 1360-bar Space Truss with Optimized Supports

6.2.3 1728-bar space truss

6.2.3.1 General Properties

Location	:	Dubai / Al-Andulus & Al-Riggae Neighbourhood Center
Main Dimensions	:	43.2m x 28.5m
Area	:	1231m^2
Critical span	:	28.5m
Module size	:	2.40m x 2.40m
Module height	:	2.08m
Number of members	:	1728
Number of nodes	:	463
Number of supports	:	10
Column sections	:	100cm x 100cm (6 of 10)
		80cm x 80cm (4 of 10)
Column lengths	:	8.5m

6.2.3.2 Loads

L1: Dead Load	:	Own weight
L2: Purlins and Claddings Load	:	130 kg/m^2
L3: Service Load	:	0 kg/m^2
L4: Live/Snow Load	:	0 kg/m^2
L5: Wind Load (left to right)	:	100 kg/m^2
L6: Wind Load (right to left)	:	100 kg/m^2
L7: Wind Load (behind to front)	:	100 kg/m^2
L8: Wind Load (front to behind)	:	100 kg/m^2
L9: Wind Load (bottom to top)	:	100 kg/m^2
L10: Wind Load (top to bottom)	:	0 kg/m^2
L11: Earthquake (x-dir)	:	Earthquake free zone
L12: Earthquake (y-dir)	:	Earthquake free zone
L13: Temperature Difference	:	±35 C

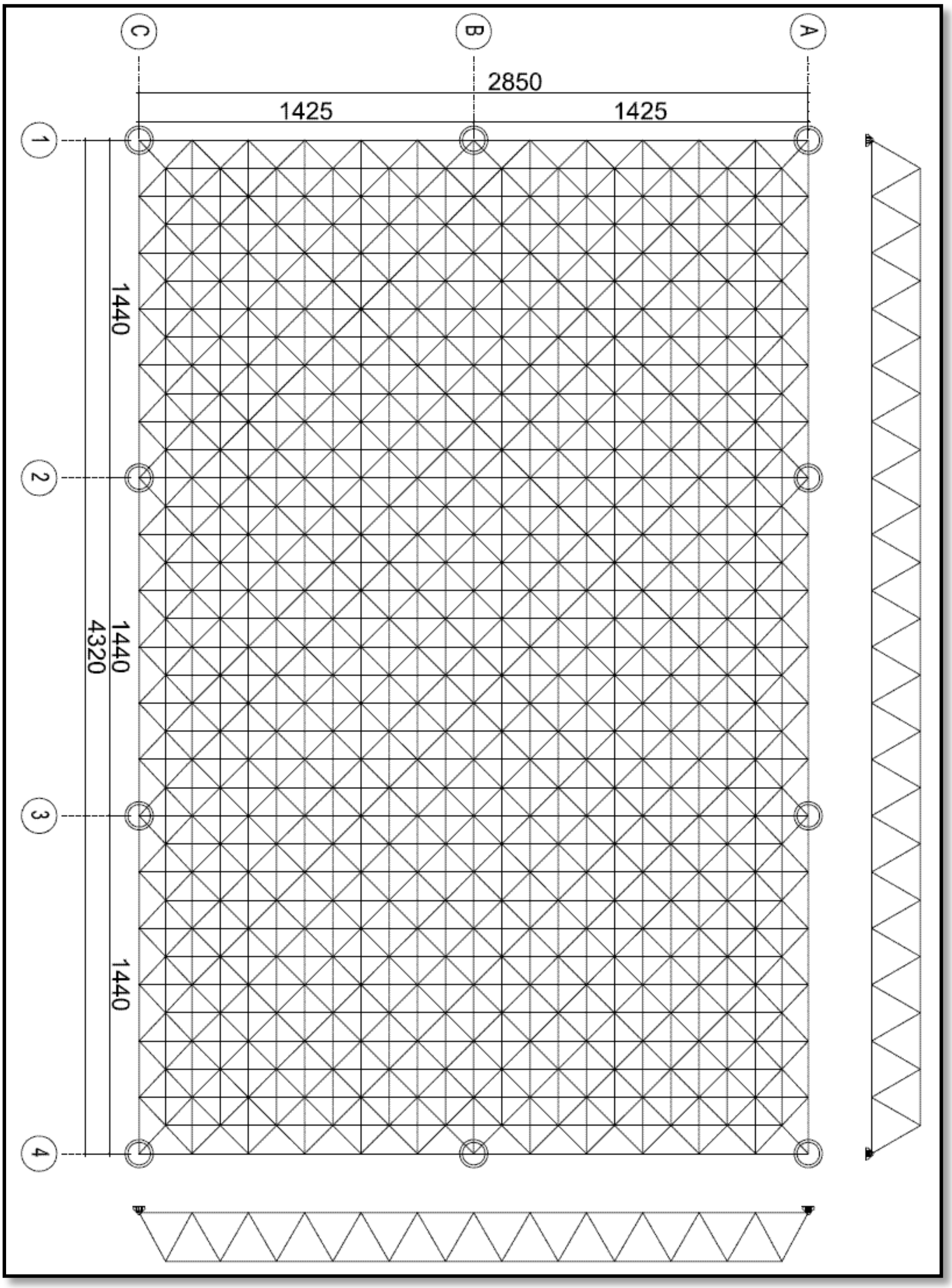

Figure –15: Plan View of 1728-bar Space Truss with Original Supports

6.2.3.3 Load Combinations

Load combinations used in 1728-bar space truss designs for Turkish Specification are given in Table 6.9. The combinations including earthquake and live loads are removed in accordance with the initial treatment of the problem with conventional design process. Load factor of purlins and claddings load was taken 0,8. Therefore, combinations including this load are modified for designs of both specifications similar to conventional design.

Table 6.9: Load Combinations for 1728-bar space truss

Comb. No.	LOAD CASE												
	L1	**L2**	**L3**	**L4**	**L5**	**L6**	**L7**	**L8**	**L9**	**L10**	**L11**	**L12**	**L13**
1	1	0,8	0	0	0	0	0	0	0	0	0	0	0
2	1	0,8	0	0	0,8	-0,4	-0,4	-0,4	0.4	0	0	0	0
3	1	0,8	0	0	-0,4	0,8	-0,4	-0,4	0.4	0	0	0	0
4	1	0,8	0	0	-0,4	-0,4	0,8	-0,4	0.4	0	0	0	0
5	1	0,8	0	0	-0,4	-0,4	-0,4	0,8	0.4	0	0	0	0
6	1	0,8	0	0	0	0	0	0	0,8	0	0	0	0
7	1	0,8	0	0	0	0	0	0	0	0	0	0	-1
8	1	0,8	0	0	0	0	0	0	0	0	0	0	1

6.2.3.4 Profile List

Profile list of pipe sections used in 1728-bar space truss is given in Table 6.10.

Table 6.10: Profile List Used in 1728-bar Space Truss

Outer Diameter (mm)	Thickness (mm)	Grade
48.3	3.00	St37
60.3	3.00	St37
60.3	3.40	St37
76.1	3.40	St37
88.9	3.76	St37
114.3	4.05	St37
114.3	4.50	St37
139.7	4.50	St37
159.0	4.50	St37
219.1	4.50	St37
219.1	6.00	St37
219.1	6.00	St52
219.1	7.00	St52
219.1	11.00	St52

6.2.3.5 Results

Results obtained by different optimization models are shown in Tables 6.11 and 6.12. Size optimization reduces total weight of the structure by 12.0%. Simultaneous optimization of size and supports results in 1.4% more reduction and accumulates to 13.4% reduction. Restraint conditions of supports in optimum structure are shown in Figure-16. By introducing elevation optimization simultaneously with size and support optimization, the reduction in weight reaches to 13.6%. Height of the structure in intial design, 2.08m changes to 1.89m after elevation optimization.

Table 6.11: Weight and Displacement Ratios of 1728-bar Space Truss before and after Optimization

		FrameCAD	OFES		
			Size	Size+Supp	Size+Supp+Elev
Weight (kg)	Pipe	18237	17270	16964	16852
	Bolt	859	452	453	466
	Nut	368	271	267	280
	Conic	908	830	795	852
	Sphere	2018	890	900	903
	TOTAL	**22390**	**19713**	**19379**	**19353**
Displacement / Span Ratio		**1/455**	**1/437**	**1/425**	**1/387**

Table 6.12: Reduction Percents in Weight of 1728-bar Space Truss after Optimization

	OFES		
	Size	Size+Supp	Size+Supp+Elev
Pipe	5.3	7.0	7.6
Bolt	47.4	47.3	45.8
Nut	26.4	27.4	23.9
Conic	8.6	12.4	6.2
Sphere	55.9	55.4	55.3
TOTAL	**12.0**	**13.4**	**13.6**

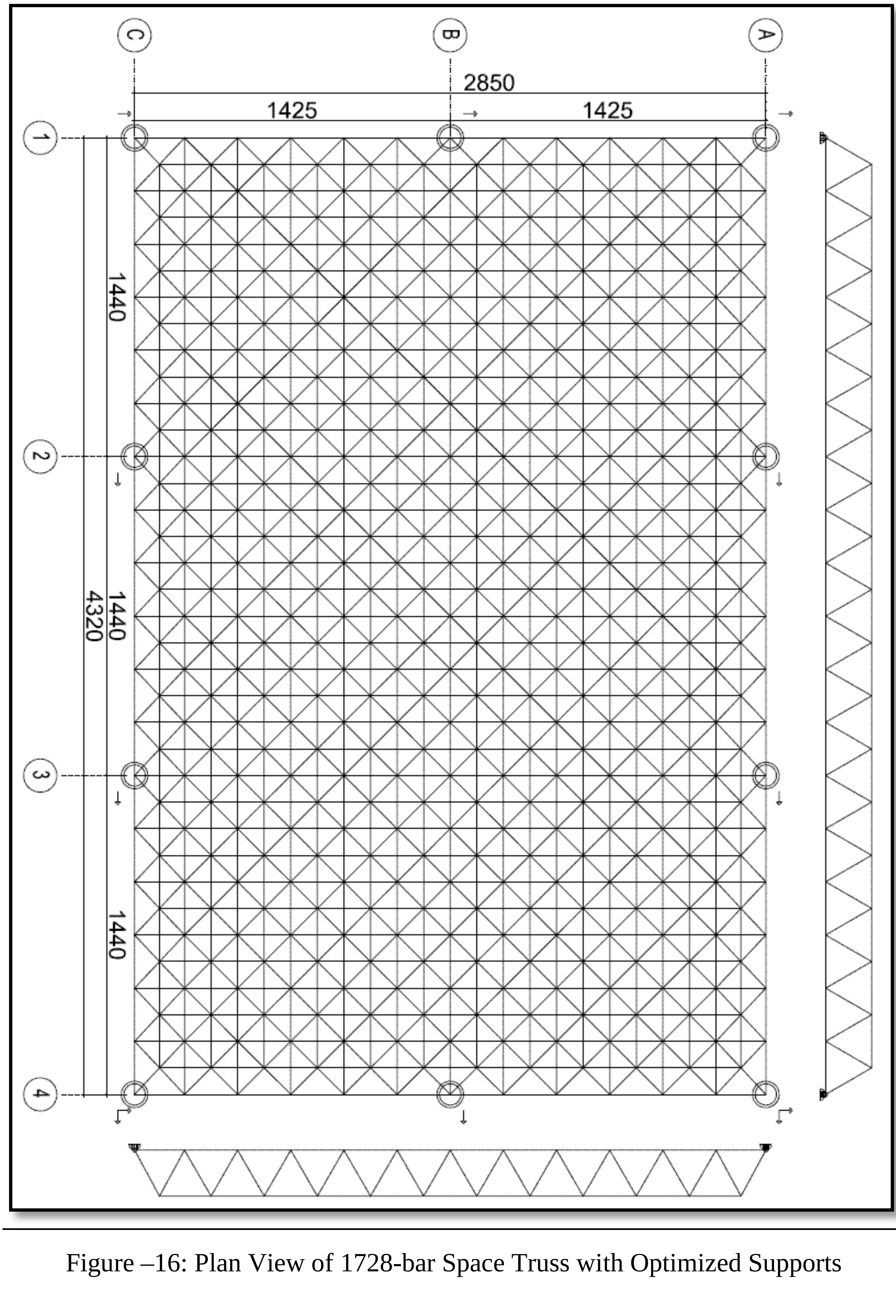

Figure –16: Plan View of 1728-bar Space Truss with Optimized Supports

6.2.4 2726-bar space truss

6.2.4.1 General Properties

Location	:	Northern Cyprus / Ayşin Karaderi Drink Factory
Main Dimensions	:	54.5m x 36.1m
Area	:	$1969m^2$
Critical span	:	35.5m
Module size	:	2.50m x 2.50m
Module height	:	2.50m
Number of members	:	2726
Number of nodes	:	720
Number of supports	:	34
Column sections	:	60cm x 60cm (4 of 34)
		80cm x 30cm (30 of 34)
Column lengths	:	5.6m

6.2.4.2 Loads

L1: Dead Load	:	Own weight
L2: Purlins and Claddings Load	:	25 kg/m^2
L3: Service Load	:	10 kg/m^2
L4: Live/Snow Load	:	60 kg/m^2
L5: Wind Load (left to right)	:	110 kg/m^2
L6: Wind Load (right to left)	:	110 kg/m^2
L7: Wind Load (behind to front)	:	110 kg/m^2
L8: Wind Load (front to behind)	:	110 kg/m^2
L9: Wind Load (bottom to top)	:	110 kg/m^2
L10: Wind Load (top to bottom)	:	110 kg/m^2
L12: Earthquake (x-dir)	:	Region=II / R=3 / I=1.2 / S(T)=2.5
L12: Earthquake (y-dir)	:	Region=II / R=3 / I=1.2 / S(T)=2.5
L13: Temperature Difference	:	±30 C

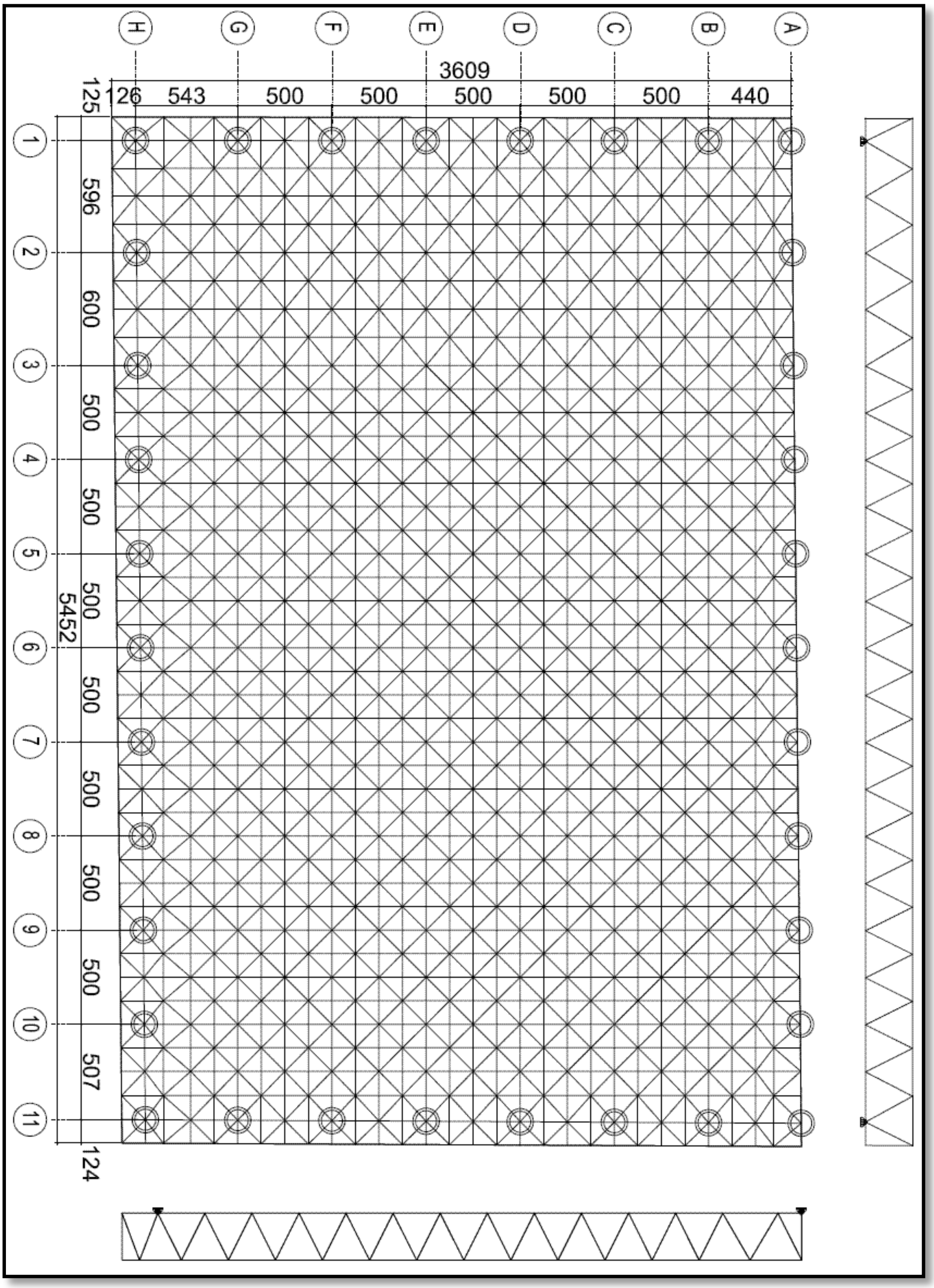

Figure –17: Plan View of 2726-bar Space Truss with Original Supports

6.2.4.3 Load Combinations

Load combinations used in 2726-bar space truss designs for Turkish Specification are given in Table 6.13.

Table 6.13: Load Combinations for 2726-bar space truss

Comb. No.	LOAD CASE												
	L1	L2	L3	L4	L5	L6	L7	L8	L9	L10	L11	L12	L13
1	1	1	1	0	0	0	0	0	0	0	0	0	0
2	1	1	1	1	0	0	0	0	0	0	0	0	0
3	1	1	1	0,5	0,8	-0,4	-0,4	-0,4	0	0	0	0	0
4	1	1	1	0,5	-0,4	0,8	-0,4	-0,4	0	0	0	0	0
5	1	1	1	0,5	-0,4	-0,4	0,8	-0,4	0	0	0	0	0
6	1	1	1	0,5	-0,4	-0,4	-0,4	0,8	0	0	0	0	0
7	1	1	1	0,5	0	0	0	0	0,8	0	0	0	0
8	1	1	1	0,5	0	0	0	0	0	-0.8	0	0	0
9	1	1	1	0,5	0	0	0	0	0	0	1	0	0
10	1	1	1	0,5	0	0	0	0	0	0	-1	0	0
11	1	1	1	0,5	0	0	0	0	0	0	0	1	0
12	1	1	1	0,5	0	0	0	0	0	0	0	-1	0
13	1	1	1	1	0	0	0	0	0	0	0	0	-1
14	1	1	1	0	0	0	0	0	0	0	0	0	1

6.2.4.4 Profile List

Profile list of pipe sections used in 2726-bar space truss is given in Table 6.14.

Table 6.14: Profile List Used in 2726-bar Space Truss

Outer Diameter (mm)	Thickness (mm)	Grade
48.3	3.00	St37
60.3	3.00	St37
60.3	3.40	St37
76.1	3.40	St37
88.9	3.76	St37
114.3	4.05	St37
114.3	4.50	St37
139.7	4.50	St37
159.0	4.50	St37
219.1	4.50	St37
219.1	6.00	St37
219.1	6.00	St52
219.1	7.00	St52
219.1	11.00	St52

6.2.4.5 Results

The results obtained by different optimization models are shown in Tables 6.15 and 6.16. Size optimization improves solution of FrameCAD by decreasing 11.7% of the total weight. Simultaneous size and support optimization reduce total weight by 14.1%. Support details of the optimized structure are given in Figure-18. Simultaneous size, support, and elevation optimization results in a 17.1% reduction by changing height of initial design from 2.50m to 2.33m. In this example, contribution of elevation optimization in reduction of total weight is in excess of ones in previous examples. The reason for that is incorrect selection of height in conventional design.

Table 6.15: Weight and Displacement Ratios of 2726-bar Space Truss before and after Optimization

		FrameCAD	OFES		
			Size	Size+Supp	Size+Supp+Elev
Weight (kg)	Pipe	31029	29899	29226	28694
	Bolt	1366	693	677	692
	Nut	649	430	408	421
	Conic	1638	1360	1315	1355
	Sphere	6080	3594	3379	2610
	TOTAL	**40762**	**35976**	**35005**	**33772**
Displacement / Span Ratio		**1/490**	**1/450**	**1/464**	**1/427**

Table 6.16: Reduction Percents in Weight of 2726-bar Space Truss after Optimization

	OFES		
	Size	Size+Supp	Size+Supp+Elev
Pipe	3.6	5.8	7.5
Bolt	49.3	50.4	49.3
Nut	33.7	37.1	35.1
Conic	17.0	19.7	17.3
Sphere	40.9	44.4	57.1
TOTAL	**11.7**	**14.1**	**17.1**

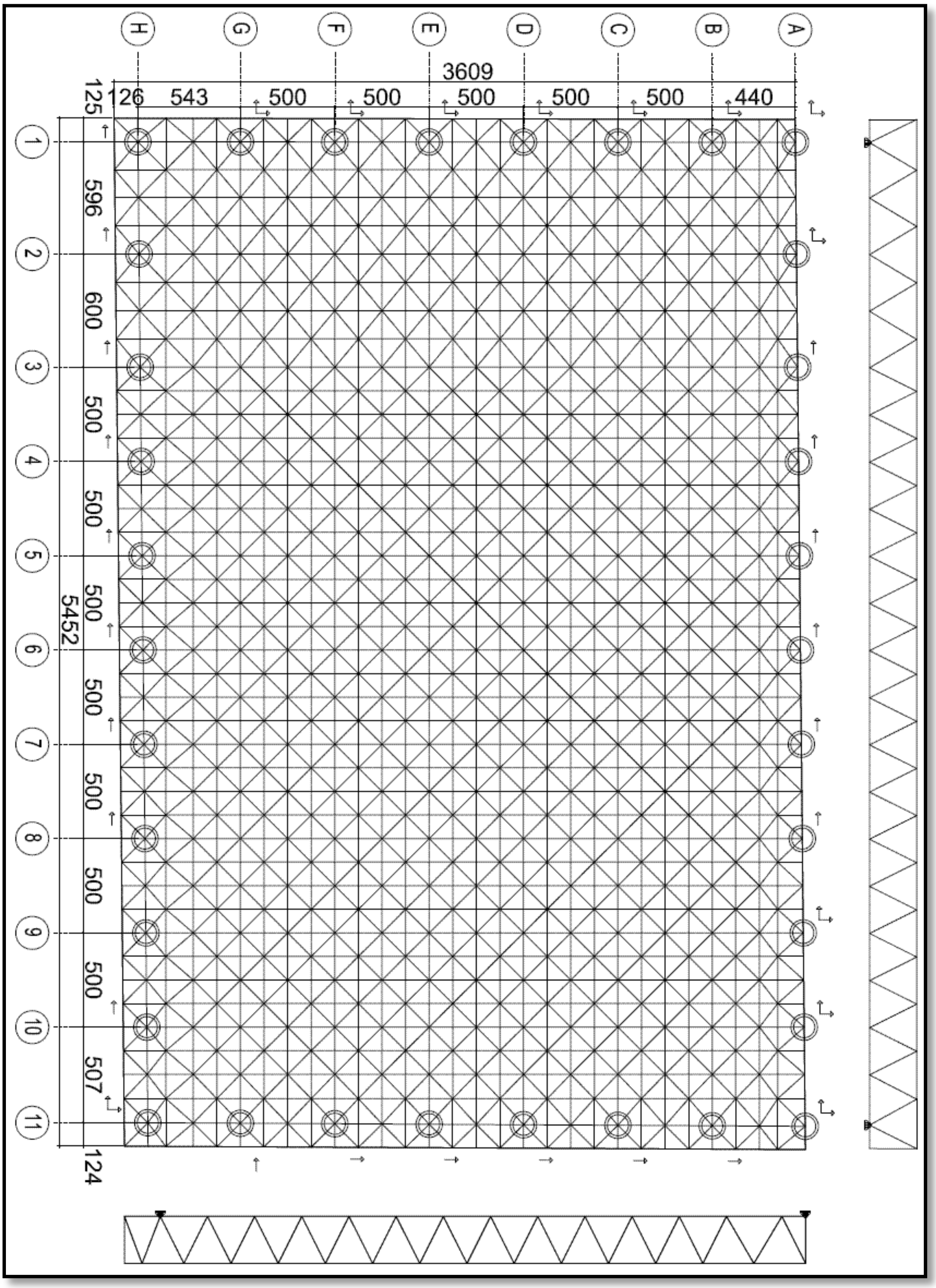

Figure –18: Plan View of 2726-bar Space Truss with Optimized Supports

6.2.5 3860-bar space truss

6.2.5.1 General Properties

Location	: İzmir / Krone Trailer Factory
Main Dimensions	: 60.1m x 55.4m
Area	: $3328m^2$
Critical span	: 20.0m
Module size	: 2.86m x 2.67m
Module height	: 1.90m
Number of members	: 3860
Number of nodes	: 1009
Number of supports	: 32
Column sections	: 70cm x 70cm
Column lengths	: 14.0m

6.2.5.2 Loads

L1: Dead Load	: Own weight
L2: Purlins and Claddings Load	: 20 kg/m^2
L3: Service Load	: 50 kg/m^2
L4: Live/Snow Load	: 75 kg/m^2
L5: Wind Load (left to right)	: 110 kg/m^2
L6: Wind Load (right to left)	: 110 kg/m^2
L7: Wind Load (behind to front)	: 110 kg/m^2
L8: Wind Load (front to behind)	: 110 kg/m^2
L9: Wind Load (bottom to top)	: 110 kg/m^2
L10: Wind Load (top to bottom)	: 110 kg/m^2
L12: Earthquake (x-dir)	: Region=I / R=3 / I=1.2 / S(T)=2.5
L12: Earthquake (y-dir)	: Region=I / R=3 / I=1.2 / S(T)=2.5
L13: Temperature Difference	: ±30 C

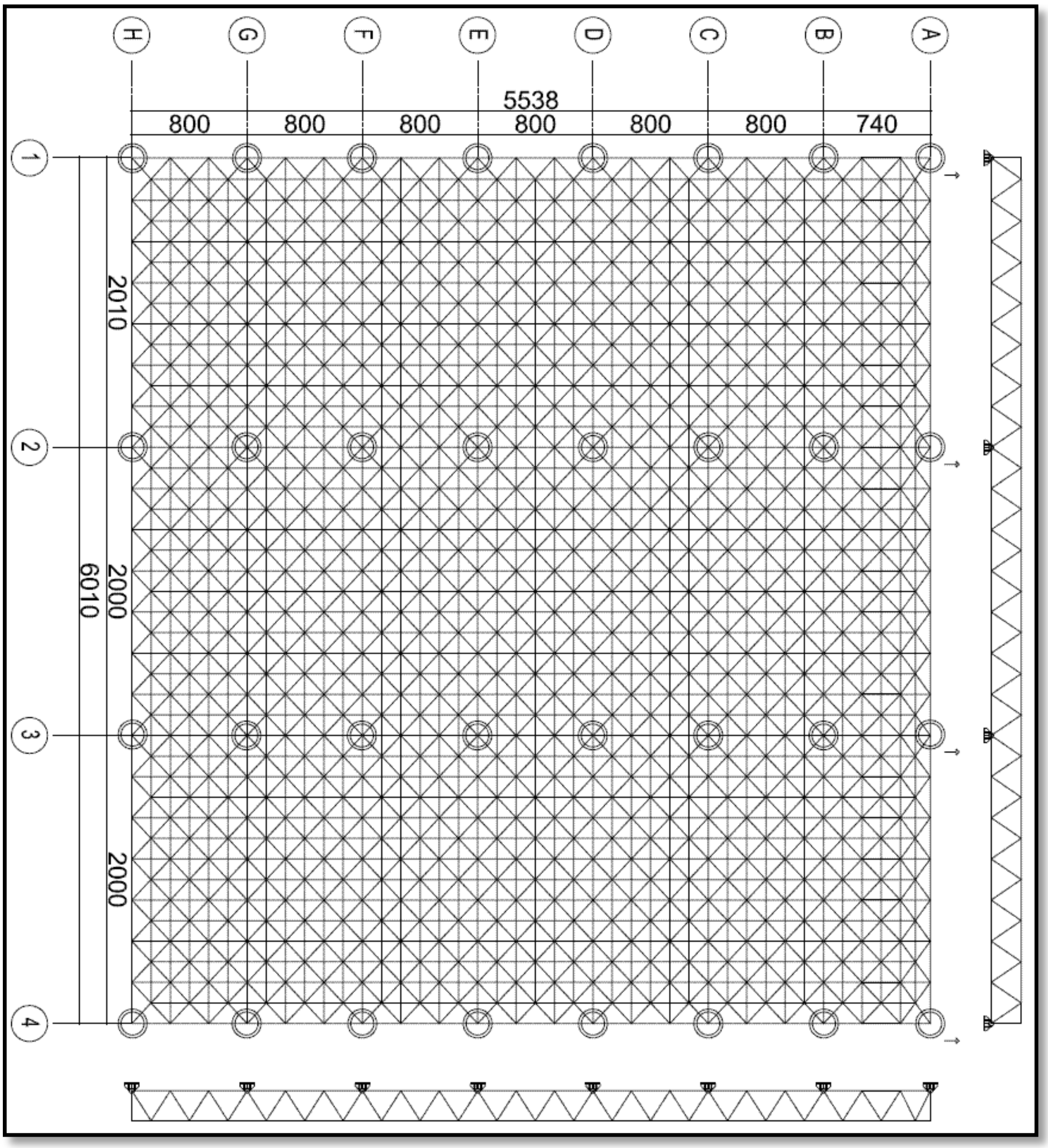

Figure –19: Plan View of 3860-bar Space Truss with Original Supports

6.2.5.3 Load Combinations

Load combinations used in 3860-bar space truss designs for Turkish Specification are given in Table 6.17.

Table 6.17: Load Combinations for 3860-bar space truss

Comb. No.	LOAD CASE												
	L1	L2	L3	L4	L5	L6	L7	L8	L9	L10	L11	L12	L13
1	1	1	1	0	0	0	0	0	0	0	0	0	0
2	1	1	1	1	0	0	0	0	0	0	0	0	0
3	1	1	1	0,5	0,8	-0,4	-0,4	-0,4	0	0	0	0	0
4	1	1	1	0,5	-0,4	0,8	-0,4	-0,4	0	0	0	0	0
5	1	1	1	0,5	-0,4	-0,4	0,8	-0,4	0	0	0	0	0
6	1	1	1	0,5	-0,4	-0,4	-0,4	0,8	0	0	0	0	0
7	1	1	1	0,5	0	0	0	0	0,8	0	0	0	0
8	1	1	1	0,5	0	0	0	0	0	-0.8	0	0	0
9	1	1	1	0,5	0	0	0	0	0	0	1	0	0
10	1	1	1	0,5	0	0	0	0	0	0	-1	0	0
11	1	1	1	0,5	0	0	0	0	0	0	0	1	0
12	1	1	1	0,5	0	0	0	0	0	0	0	-1	0
13	1	1	1	1	0	0	0	0	0	0	0	0	-1
14	1	1	1	0	0	0	0	0	0	0	0	0	1

6.2.5.4 Profile List

Profile list of pipe sections used in 3860-bar space truss is given in Table 6.18.

Table 6.18: Profile List Used in 3860-bar Space Truss

Outer Diameter (mm)	Thickness (mm)	Grade
48.3	3.00	St37
60.3	3.00	St37
60.3	3.40	St37
76.1	3.40	St37
88.9	3.76	St37
114.3	4.05	St37
114.3	4.50	St37
139.7	4.50	St37
159.0	4.50	St37
219.1	4.50	St37
219.1	6.00	St37
219.1	6.00	St52
219.1	7.00	St52
219.1	11.00	St52

6.2.5.5 Results

Results of different optimization models are shown in Table 6.19, and Table 6.20. Size optimization results in 5.1% reduction, simultaneous size and support optimization results in 5.8%, and simultaneous optimization of all size, support and elevation variables results in 6.5% reduction in total weight of structure compared to conventional FrameCAD solution. Optimized support conditions are shown in Figure-20. Initial height of structure, which is equal to 1.90m, becomes 1.70m after elevation optimization.

Table 6.19: Weight and Displacement Ratios of 3860-bar Space Truss before and after Optimization

		FrameCAD	OFES		
			Size	**Size+Supp**	**Size+Supp+Elev**
Weight (kg)	**Pipe**	39130	38595	38368	37798
	Bolt	1709	882	870	917
	Nut	626	486	485	506
	Conic	1570	1526	1502	1549
	Sphere	2823	2012	1961	2125
	TOTAL	**45858**	**43501**	**43186**	**42895**
Displacement / Span Ratio		**1/651**	**1/643**	**1/641**	**1/560**

Table 6.20: Reduction Percents in Weight of 3860-bar Space Truss after Optimization

	OFES		
	Size	**Size+Supp**	**Size+Supp+Elev**
Pipe	1.4	1.9	3.4
Bolt	48.4	49.1	46.3
Nut	22.4	22.5	19.2
Conic	2.8	4.3	1.3
Sphere	28.7	30.5	24.7
TOTAL	**5.1**	**5.8**	**6.5**

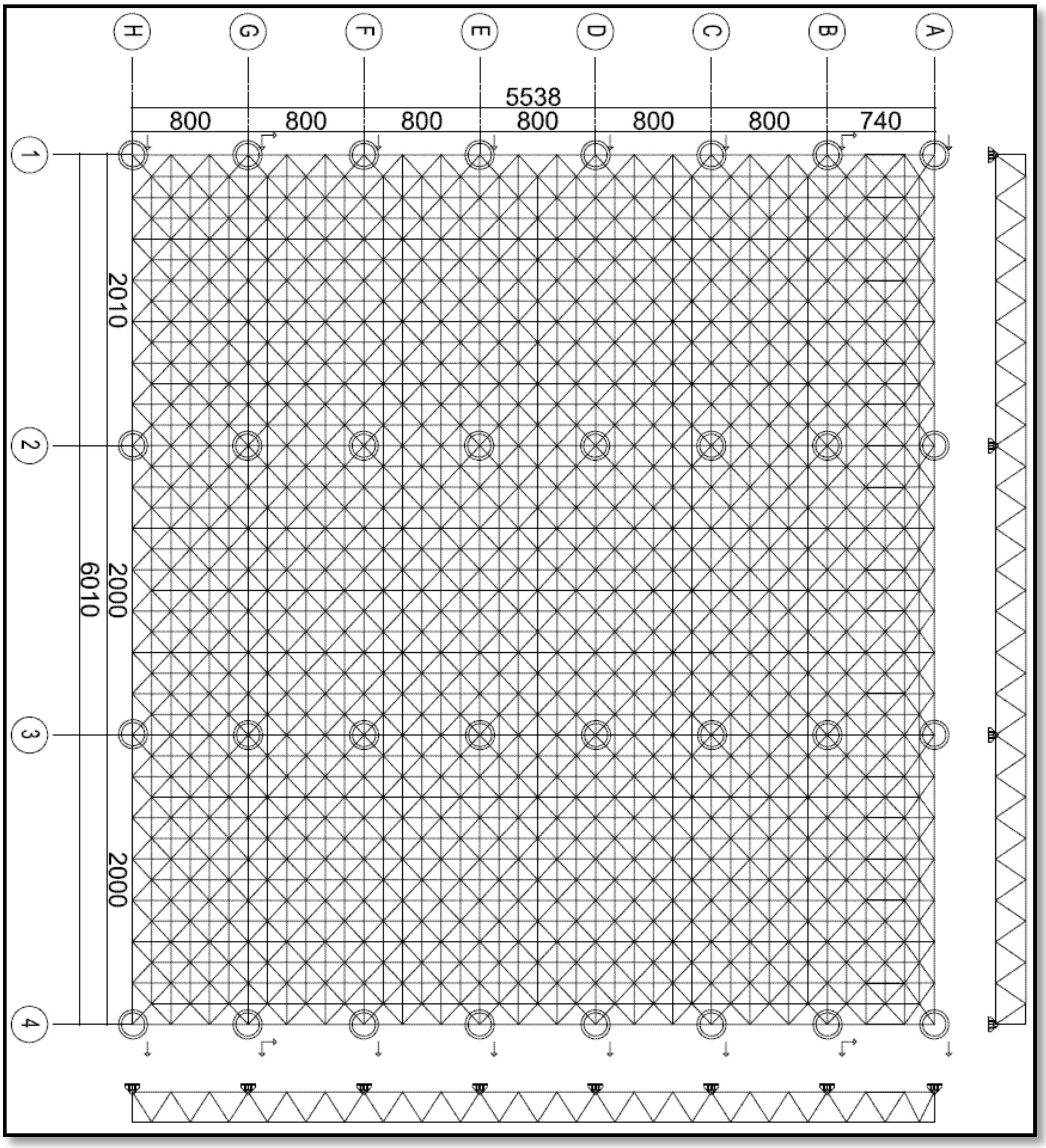

Figure –20: Plan View of 3860-bar Space Truss with Optimized Supports

6.2.6 4412-bar space truss

6.2.6.1 General Properties

Location	:	İzmir / Sunel Tobacco Warehouse
Main Dimensions	:	70.7m x 67.4m
Area	:	4168m^2
Critical span	:	23.6m
Module size	:	2.95m x 2.82m
Module height	:	1.60m
Number of members	:	4412
Number of nodes	:	1153
Number of supports	:	38
Column sections	:	70cm x 70cm
Column lengths	:	6.25m

6.2.6.2 Loads

L1: Dead Load	:	Own weight
L2: Purlins and Claddings Load	:	15 kg/m^2
L3: Service Load	:	10 kg/m^2
L4: Live/Snow Load	:	75 kg/m^2
L5: Wind Load (left to right)	:	80 kg/m^2
L6: Wind Load (right to left)	:	80 kg/m^2
L7: Wind Load (behind to front)	:	80 kg/m^2
L8: Wind Load (front to behind)	:	80 kg/m^2
L9: Wind Load (bottom to top)	:	80 kg/m^2
L10: Wind Load (top to bottom)	:	0 kg/m^2
L12: Earthquake (x-dir)	:	Region=I / R=3 / I=1.2 / S(T)=2.5
L12: Earthquake (y-dir)	:	Region=I / R=3 / I=1.2 / S(T)=2.5
L13: Temperature Difference	:	±30 C

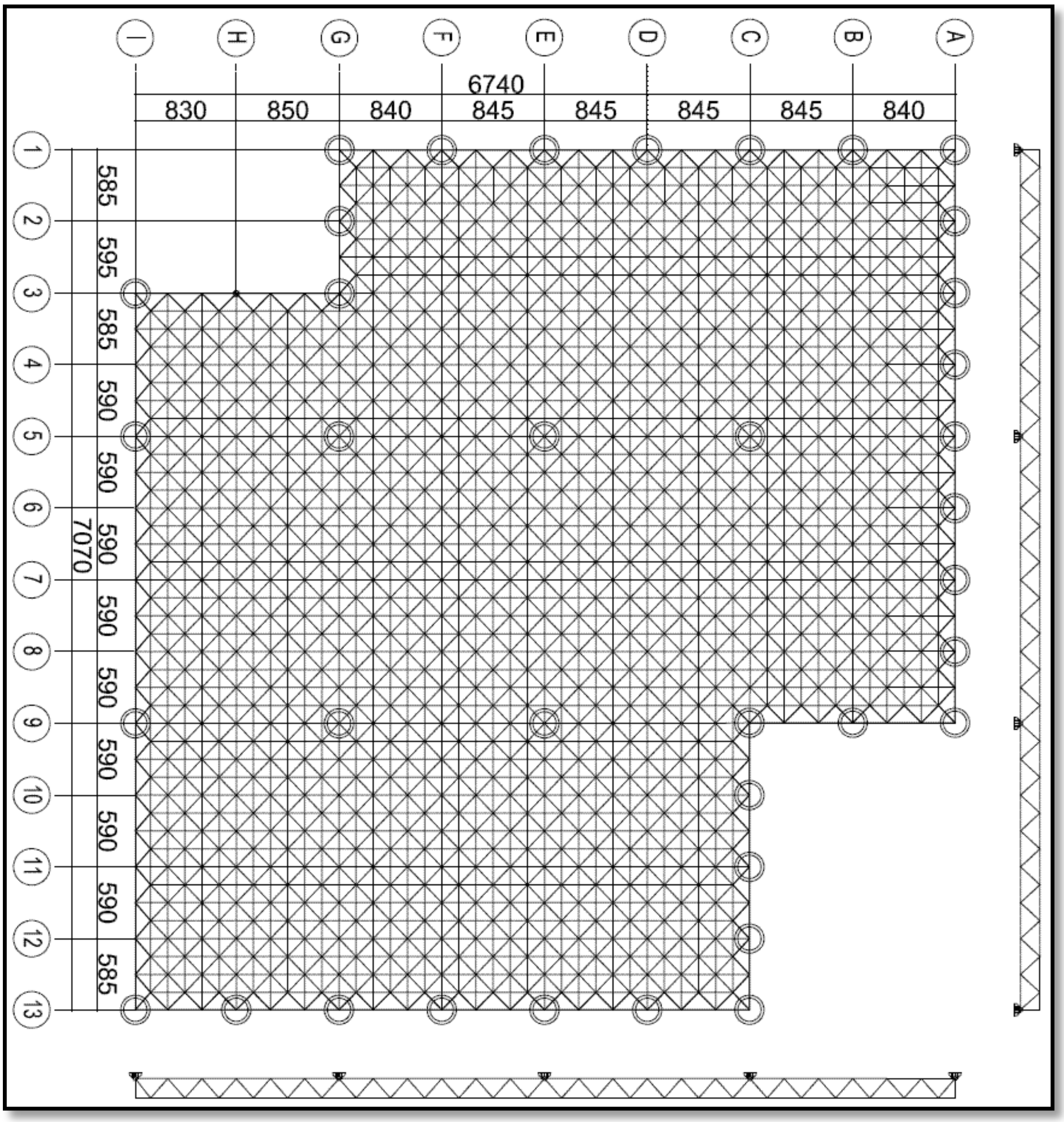

Figure –21: Plan View of 4412-bar Space Truss with Original Supports

6.2.6.3 Load Combinations

Load combinations used in 4412-bar space truss designs for Turkish Specification are given in Table 6.21.

Table 6.21: Load Combinations for 4412-bar space truss

Comb. No.	LOAD CASE												
	L1	L2	L3	L4	L5	L6	L7	L8	L9	L10	L11	L12	L13
1	1	1	1	0	0	0	0	0	0	0	0	0	0
2	1	1	1	1	0	0	0	0	0	0	0	0	0
3	1	1	1	1	0,8	-0,4	-0,4	-0,4	0,4	0	0	0	0
4	1	1	1	1	-0,4	0,8	-0,4	-0,4	0,4	0	0	0	0
5	1	1	1	1	-0,4	-0,4	0,8	-0,4	0,4	0	0	0	0
6	1	1	1	1	-0,4	-0,4	-0,4	0,8	0,4	0	0	0	0
7	1	1	1	1	0	0	0	0	0	0	1	0,3	0
8	1	1	1	1	0	0	0	0	0	0	1	-0,3	0
9	1	1	1	1	0	0	0	0	0	0	-1	0,3	0
10	1	1	1	1	0	0	0	0	0	0	-1	-0,3	0
11	1	1	1	1	0	0	0	0	0	0	0,3	1	0
12	1	1	1	1	0	0	0	0	0	0	-0,3	1	0
13	1	1	1	1	0	0	0	0	0	0	0,3	-1	0
14	1	1	1	1	0	0	0	0	0	0	-0,3	-1	0
15	1	1	1	1	0	0	0	0	0	0	0	0	-1
16	1	1	1	0	0	0	0	0	0	0	0	0	1

6.2.6.4 Profile List

Profile list of pipe sections used in 4412-bar space truss is given in Table 6.22.

6.2.6.5 Results

The results obtained by different optimization models are shown in Tables 6.23 and 6.24. Size optimization results in 10.2%, size and support optimization results in 12.1%, and simultaneous optimization of size, support and elevation variables results in 12.3% reduction in total weight of structure. Figure-22 shows restraint details of the supports after optimization. Initial height of structure changes from 1.60m to 1.70m after elevation optimization.

Table 6.22: Profile List Used in 4412-bar Space Truss

Outer Diameter (mm)	Thickness (mm)	Grade
48.3	2.50	St37
48.3	3.00	St37
60.3	2.50	St37
60.3	3.00	St37
60.3	3.40	St37
76.1	3.40	St37
88.9	3.76	St37
114.3	4.05	St37
114.3	4.50	St37
139.7	4.50	St37
159.0	4.50	St37
219.1	4.50	St37
219.1	6.00	St37
219.1	6.00	St52
219.1	7.00	St52
219.1	11.00	St52

Table 6.23: Weight and Displacement Ratios of 4412-bar Space Truss before and after Optimization

		FrameCAD	OFES		
			Size	Size+Supp	Size+Supp+Elev
Weight (kg)	Pipe	43603	41407	40538	40547
	Bolt	2047	1049	1041	1024
	Nut	829	612	605	595
	Conic	2182	1977	1884	1835
	Sphere	4047	2284	2263	2231
	TOTAL	**52708**	**47329**	**46331**	**46232**
Displacement / Span Ratio		**1/491**	**1/481**	**1/492**	**1/523**

Table 6.24: Reduction Percents in Weight of 4412-bar Space Truss after Optimization

	OFES		
	Size	**Size+Supp**	**Size+Supp+Elev**
Pipe	5.0	7.0	7.0
Bolt	48.8	49.1	50.0
Nut	26.2	27.0	28.2
Conic	9.4	13.7	15.9
Sphere	43.6	44.1	44.9
TOTAL	**10.2**	**12.1**	**12.3**

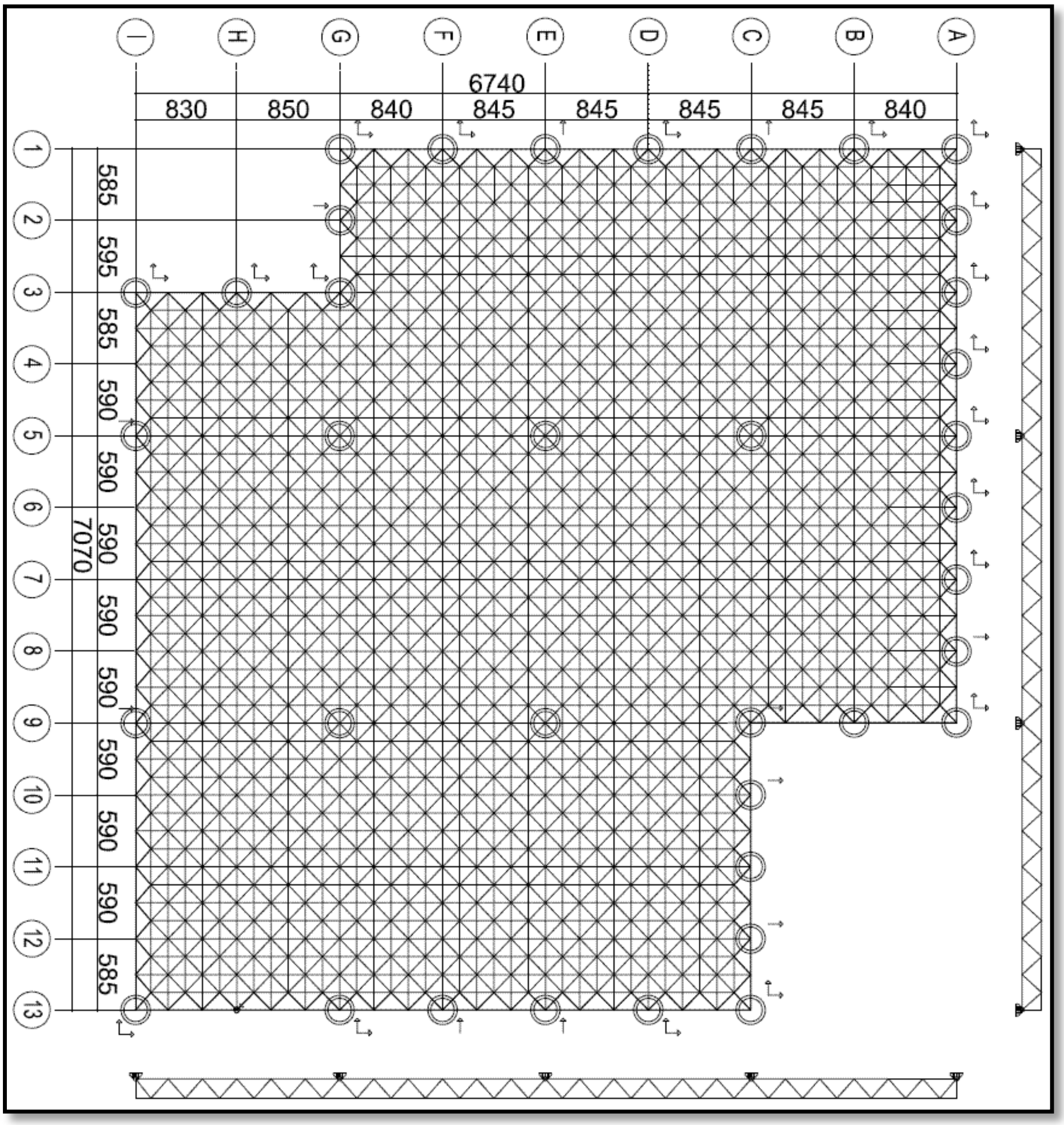

Figure –22: Plan View of 4412-bar Space Truss with Optimized Supports

6.3 Discussion of Results

As compared to conventional FrameCAD solution, average percentages of weight reductions obtained with optimum design process are given in Table 6.25 for every part of structure. These values are evaluated by using six space truss examples having different number of members according to TS648. It is seen that an average weight reduction of 13.9% can be achieved by optimizing structural system using evolution strategies method. Approximately 83% of this reduction, which is equal to 11.6%, is obtained by size optimization. By introducing support optimization, this ratio increases to 13.2%, indicating that an additional weight reduction of 1.6% is attained when supports are optimized with member sizes. If elevation optimization is also employed together with size and support optimizations simultaneously, 0.7% more reduction is provided and average 13.9% reduction can be provided in total weight of structures

Best reduction is obtained in weight of spheres with an average 50.3%. The difference in evaluation of sphere diameters has an important contribution to this ratio. Minimum average reduction is obtained in weight of pipe members with an average 8.2%.

Table 6.25: Average Reduction Percents in Weight of Space Trusses after Optimization

	OFES		
	Size	**Size+Supp**	**Size+Supp+Elev**
Pipe	5.9	7.5	8.2
Bolt	39.6	40.0	39.6
Nut	24.2	25.1	24.1
Conic	10.4	13.3	11.5
Sphere	48.2	49.2	50.3
TOTAL	**11.6**	**13.2**	**13.9**

CHAPTER 7

CONCLUSIONS

7.1 Overview of the Thesis

This work presents a comprehensive study about double-layer grids, which are the most common type of space trusses having two layers of horizontal members connected each other with diagonal members. All parts of double-layer grids are defined, their design requirements and calculation of allowable capacities are explained in details. In other words, almost all relative information required to design double layer grids are available in scope of this study.

Traditional design methods without using computational effort are hard to be implemented and result sections having sizes over than required. Therefore, an optimization routine is developed to reduce weight of structures. Evolution strategies technique is preferred due to the reason that it converges to optimum result in shorter time compared to other similar techniques. It is a randomized optimization method, which simulates the biological evolution. General properties of evolution strategies are summarized, operators used in evolution process are defined and mathematically formulized.

An algorithm is developed to computerize optimization process. A software named as OFES, which can design all members and connection parts of double-layer grids by minimizing weight of structures, is modified to implement this algorithm. Unlike many computer programs optimizing truss systems in literature, this program can be used in practice easily. It gives a complete calculation report including all responses and allowable responses for each element of double-layer grids. Connection details and size of elements required for manufacturing are also available in this report. In addition, a shop drawing showing the location of members and spheres in a plan view is also given as an output to make assembling easily.

Six examples having different number of members changing between 792 and 4412 are optimized for minimization of total weight. Size of members, restraint conditions of supports and height of structures are used as design variables. They are optimized both separately and simultaneously to identify their contributions to reduction of weight. Results are presented in detail and compared with each other and designs obtained by using FrameCAD.

It is seen that lighter double-layer grid systems applicable in real-life practice can be designed by using evolution strategies method. An average 13.9% reduction is obtained by using all size, support and support optimization models. 83% of this reduction is obtained by using only size optimization model. In addition, it is concluded that using “iterative design method” instead of randomly created initial population used in conventional implementation of evolution strategies make optimization process faster.

7.2 Recommendation for Future Work

This study provides an applicable optimization process for weight minimization of double-layer grids, which gives good results compared to non-optimum real life structures. However, it can be developed to obtain better solutions. One issue to be developed is using cost minimization instead of weight minimization as objective function of optimization routine. Minimum weight does not mean minimum cost in this type of structures. For example, a heavier structure having smaller number of member types can be more economical than lighter structure having larger number of different member types. Increasing the number of member types make both manufacturing and assembling of the structure harder and more expensive.

Nevertheless, preparing an objective function for cost minimization is not an easy task. Every stage of assembling and manufacturing has to be considered and formulized mathematically. Unfortunately, these stages have differences changing from one company to another. Therefore, user defined formulas must be preferred instead of generalized ones.

Another progress can be provided by including weight of connection elements, spheres, nuts, bolts, and conics, in objective function. In this study, they are included in detailing stage after finishing of optimization process. In this case, weights of members are minimized, but this does not mean that total weight of structure is also minimized. Sometimes by using heavier pipes, but lighter connection members, especially spheres, total weight of structure may be lower. By including connection parts in optimization process, lighter solutions can be obtained.

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