

# Optimum design of wide span cable-stayed roof structures

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**Abstract.** This paper investigates the structural response of wide span structures where the primary structural system is cable-stayed roof, and aims at optimizing the design of such structures. Finite element analysis is carried out using commercial software taking into consideration the nonlinear formulation of cables. In order to check the accuracy and capability of the present approach, several examples are presented for nonlinear analysis of cable structures, and the response is found to agree with published results. Additionally, finite element discretization and nonlinear analysis are carried out for an existing cable-stayed roof structure under all possible loading conditions. A numerical study is conducted where four different systems are suggested for the same structure: plane truss, space truss, cable-stayed plane truss and cable-stayed space truss systems. For each system, structural analysis is made under all design load combinations and optimization of design in order to reach the least possible weight and cost. The numerical results indicate the capability of the proposed approach in predicting the forces and deformations of cable structures and optimizing the design of structural element. It was concluded that the space truss and the cable-stayed space truss providing efficient performance as well as economy.

**Keywords:** cable-stayed; roof structure; steel truss; optimum design; finite elements; nonlinear analysis; elastic catenary.

## 1. Introduction

Cables structures have been used as wide span roof structures due to their various advantages such as high strength, light weight, design flexibility, cost-effective construction and innovative forms. They have application in a variety of wide span structures like exhibition halls, stadiums, sports halls, swimming pools, warehouses, factories and hangers. Cables are flexible tension members that consist of one or more groups of steel wires, strands or ropes. The tensile strength of the wires of the cable exceeds 1570 MPa and the 0.2 % proof stress is over 1180 MPa. Cable-supported structures can be sorted into two categories: cable suspended structures, where draped cables are the main supporting elements of the structure and their curvature is a major factor in the load-carrying capacity of the system; and cable-stayed structures in which cables are tension members fixed inclined to masts or pylons acting as compression members and where cables are straight with small amount of sag due to cables own weight (Buchholdt 1985).

Cables systems are widely applied in bridges, but cable supported system is a relatively new form of roof construction.

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The first cable roofs were four pavilions with hanging roofs constructed in 1896 at an exhibition in Nizhny-Novgorod, Russia, and then some minor cable roofs were built during the 1930's (Otto and Schleyer 1969). Development of cable roof structures was made between 1950's and early 1960's in the United States and Germany. The first major structure with a cable roof was the Arena in Raleigh, North Carolina, USA, built in 1952. The doubly curved saddle-shaped roof shown in Fig.1 (a) is an anticlastic cable mesh of dimensions 92m by 97m consisting of 47 prestressed cables, suspended between two parabolic concrete arches intercrossing each other. Simply suspended cable systems were also used such as Dulles International Airport near Washington, USA, built in 1963 with a rectangular floor plan 183m by 46m. Steel cables suspended from outward leaning reinforced concrete piers support a precast concrete roof, as shown in Fig. 1 (b). The roof of Burgo paper mill, Fig. 1 (c), designed by Luigi Nervi and built in 1963 at Mantua, Lombardy, Italy, is supported by cables suspended from pylons 45m high with span 163m. Other early examples are the German Pavilion in Montreal Expo, 1967 and the Olympic Park Roof designed for the 1972 Olympics in Munich, Germany (Buchholdt 1985, Irvine 1999).

Several structures were designed with the pre-tensioned straight cable system, some examples are shown in Fig. 2. The pretension cable structures might be either self-balancing where the cables forces are internally balanced in the supporting structure like the masts or ring beam, or non- self- balancing where ground anchors resist the cable forces. McCormick Place exhibition hall in Chicago, Illinois, USA constructed in 1986 over active railroad tracks, has roof consisting of steel trusses supported by steel



cables hung from 12 concrete pylons spaced 27m by 55m and rise to 18m through the building. The building width is 110m, spacing between the pylons is 55m and 27m overhangs on both sides. The Millennium Dome in London, UK, built in 2000, is a fabric clad tensioned cable structure having a diameter of 320m and a central height of 48m. The roof adopts the pre-tensioned straight cable system, where 72 tensioned steel stringer cables are arranged radially on the dome surface, stretching between a central cable ring and edge cables which are anchored to the ground. These stringers are supported at a radial spacing of between 25m and 30m by circumferential cables, which are supported by an arrangement of upper hanger and lower tie-down cables attached to the steel masts (Gonzalez Quelle 2009). The City of Manchester Stadium in Manchester, UK, constructed in 2002, employs a cable stayed system where cables are attached to the twelve masts circling the stadium with rafters and purlins for additional rigid support. The roof of the stadium in Chonju, Korea, 2002, having an area of 260m by 160m comprises cable-stayed truss structure where a prismatic inner ring steel truss acts as the primary support system to the roof, and is suspended at 28 points around the stadium by 65-85 mm front stay cables. The front and back stay cables are suspended by four 63 m high masts located at the corners of the stadium (Kim 2001). The roof of Braga Municipal Sports Stadium, Portugal, constructed in 2001, is formed by pairs of full locked coil cables spanning 202m and supporting two concrete slabs of thickness 0.25 m and length 57.3m. The roof

cables are anchored in two large beams at the top of both stands made up of concrete walls (Caetano *et al.* 2010). The stadium of Warsaw, completed in 2011, has a fully retractable composite membrane central roof supported by 60 cables (Knut and Lorenz 2013). The structural system for the gymnasium of the Gerald Ratner Athletics Center, constructed in 2003 on the campus of the University of Chicago, Illinois, USA, is a cable-stayed roof system of composite masts that are sloped, tapered, and stabilized by 15 cables (nine fore-stay cables and six back-stay cables), which support S-shaped roof girders, allowing a column-free space of about 40m by 60m (Kloiber *et al.* 2004). Due to the low bending stiffness of the cables, cables structures exhibit relatively large displacements and thus should be analyzed considering geometric nonlinearity (Buchholdt 1985). These structures are sensitive to construction procedure and being lightweight, they are also sensitive to dynamic excitations and wind loads. Design of tensile structures needs special care as errors may occur in the pre-tensioning forces distribution causing cladding damage. In cable-stayed roof, the significant elongation of the cables and deformation of the supported structure must be taken into consideration. Analysis of these systems should consider high geometric nonlinearity to ensure stability of cable structures (Toklu *et al.* 2017).

This paper investigates the structural performance of cable supported roofs of wide span halls and presents optimum design solution for such structures. Numerical modeling is made for cable-stayed roof of an existing hall and nonlinear analysis is



carried out under all possible loading conditions to study the structural behavior and evaluate its safety margin. The structural nonlinear analysis is performed by finite element method using the commercial software CSI SAP2000.V 18.2 (2017). Additionally, four different roofs including two cable-stayed systems, were suggested for the same structure, analyzed and compared. Optimization is carried out in order to reach the least possible weight.

**2. Analytical and finite element formulations for cables**

For modeling of cable structures, two methodologies have been used. The first approach is based on interpolation polynomial functions to describe the shape and displacement; as in finite element analysis. The simplest element most commonly used in the analysis of cable structures is the two-node straight bar element having only axial stiffness, and is regarded to give acceptable response for low-sag, highly stretched cables (Ozdimir 1979). Other models are the multi-node isoperimetric element which uses higher order polynomials, and curved element model with degrees of free rotation added to the nodes (Huu and Seung 2011).

The second approach uses mathematical expression or analytical function that describes the cable taking into account the effect of loading applied along the cable length. A flexible cable supported from its ends and exposed to a uniform downward gravitational force was assigned the shape of a catenary in 1691 by Bernoulli, Leibniz and Huygens (Irvine

1992). An extensible cable with small segments is shown in Fig. 3, but when  $AE \rightarrow \infty$  these segments convert to be inextensible (Tibert 1999). The inextensible catenary element may experience numerical instability and non-convergence (Andreu *et al.* 2006). Cables have a finite axial flexibility and thus are extensible in reality. The solution of the differential equilibrium equations of a stretched chain is derived satisfying Hooke's law considering vertical and horizontal equilibrium of a segment of the extensible cable as shown in Fig. 4, and applying the geometric constraint for the elastic catenary. Elastic catenary cable formulation was first presented by O'Brien and Francis (1964) and Jayaraman and Knudson (1981), it provides continuity, considers nonlinear effects accurately and requires less number of elements in cable structures modeling (Andreu *et al.* 2006, Yang and Tsay 2007). The parabolic elastic element is also used due to its simpler shape compared to catenary (Irvine 1992), and is reported to be acceptable for cables experiencing small curvatures (Huang and Ren, 2005).

Finite elements for nonlinear analysis of cable structures were presented by Thai and Kim (2011) considering the effect of self-weight, where the tangent stiffness matrix and internal force vector were obtained explicitly under static and dynamic loading. Salehi Ahmad Abad *et al.* (2013) introduced two new catenary cable elements for both material and geometric nonlinear dynamic analysis. Greco *et al.* (2014) used cable elements based on catenary solution to analyze cables with several nodes and general loads such as wind loads, snow loads

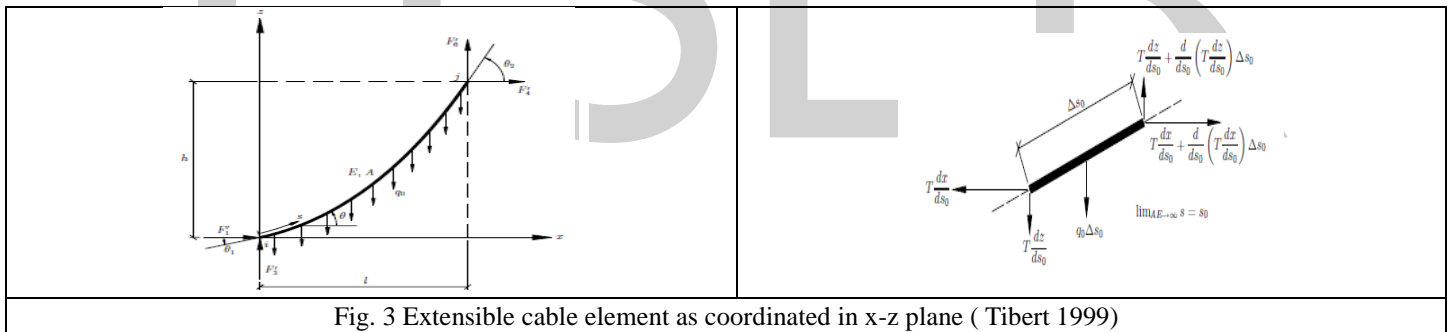


Fig. 3 Extensible cable element as coordinated in x-z plane (Tibert 1999)

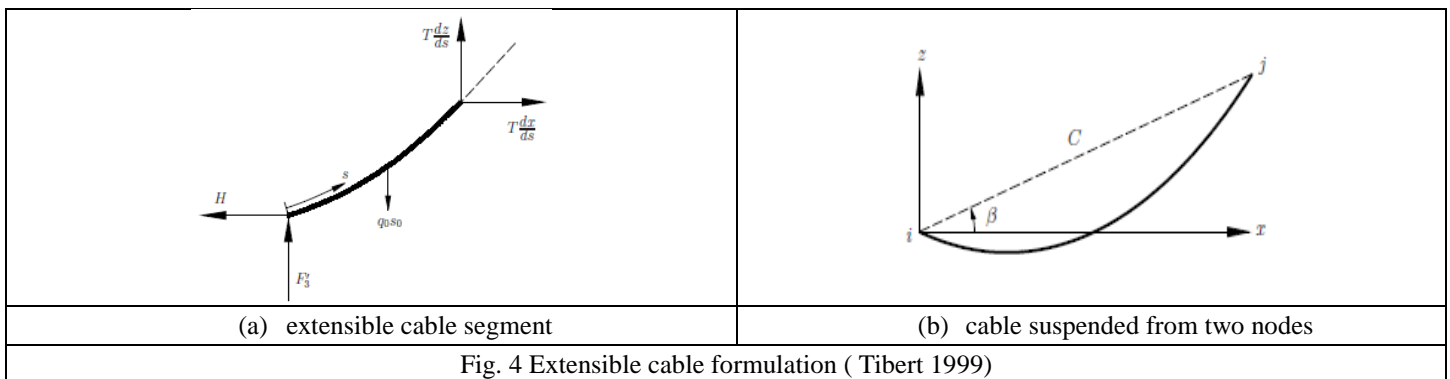


Fig. 4 Extensible cable formulation (Tibert 1999)

1992). The equilibrium equations for inextensible cable were used to derive the equation of the catenary curve assuming that the cable is free of torsional rigidity, able to sustain no compression force, inextensible ( $AE \rightarrow \infty$ ) and very flexible ( $EI \equiv 0$ ). The cable forces are thus tangential to every point of the

and cladding. Coarita and Flores (2015) proposed a mixed algorithm to simulate the interaction between cable-truss. Yan *et al.* (2015) proposed a finite element to simulate the friction in cable-strut joints by defining a virtual node. A spatial catenary cable element was presented by Vu *et al.* (2012) for nonlinear

analysis of cable-supported structures, considering the effect of self-weight and pretension. The equilibrium equation was solved by an incremental-iterative solution based on the Newton-Raphson method, and an algorithm for form-finding of cable-supported structures is proposed. Cao *et al.* (2017) extended the analytical form-finding method based on the elastic catenary theory and presented an explicit analytical iteration method for form-finding analysis of suspension bridges; the unstrained lengths are derived from nonlinear governing equations.

### 3. Adopted nonlinear analysis and optimization procedure

In this work, static nonlinear analysis is performed by finite element method using the commercial software CSI SAP2000.v.18 (2017). The element 'Cable' implemented in the software is used to model the cables. The Cable element uses elastic catenary formulation to represent the behaviour of a slender cable under its own self-weight, temperature, and strain loading. Tension-stiffening and large-deflections nonlinearity are inherently included in the formulation, in addition to slack and taut behaviours. Large displacements analysis considers the equilibrium equations in the deformed configuration of the structure. The program tracks the position of the element using an updated Lagrangian formulation. Different combinations of loads are applied; the nonlinear equations are solved iteratively by Newton-Raphson method.

Optimization process is applied in this study for getting the minimum possible weight of the structure. For optimum design of trusses, Dizangian and Ghasemi (2016) introduced amplification factor-based approach using Response Surface Method instead of than reliability analysis [26]. Artar (2016) used harmony search and genetic algorithms for minimum weight designs by selecting suitable profile sections from a specified list taken from AISC, and developed a computer program interacting with SAP2000 to obtain solution of design problems for several truss structures [27]. In the present work, optimum design is performed via SAP2000 software. Optimization is performed through assign frame section as an auto selective section through many different sections to choose the optimum section that could resist all applied forces with a minimum weight as possible. The stress constraints according to AISC and displacement constraints are considered for optimum designs.

### 4. Verification and comparative studies

In order to check the accuracy and capability of the present models, examples for evaluating proposed models are studied, and the obtained results are compared to results in published research work.

#### 4.1 Verification example 1: Single suspended cable

This verification example was previously analysed by several researchers in published work. An isolated cable is suspended between two supports at the same level with 304.8 m span and is subjected to a concentrated load of 35.586 kN. The initial shape

of cable is shown in Fig. 5, where the sag of this cable is 30.48 m at the mid span (Thai and Kim 2011, Coarita and Flores 2015). The problem data and cable properties are given in Table 1. Nonlinear static analysis is carried out using the software CSI SAP2000, the problem is modeled by two elastic catenary cable elements. The displacement of point 2 obtained numerically in this work is compared to the value given by other researchers in Table 2. The obtained numerical results for the cable deformed shape due to all loads and the axial force along the cable are shown in Fig. 6. Good agreement can be observed between the obtained and published results

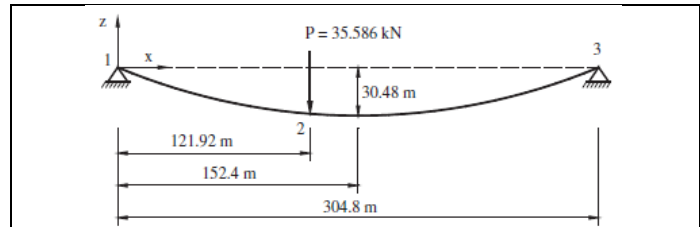


Fig. 5 Verification example 1: Geometry of cable ( Thai and Kim 2011)

Table 1 Verification example 1: Cable initial properties (Thai and Kim 2011)

Item	Data
Cross-sectional area	548.4 mm <sup>2</sup>
Elastic modulus	131000 N/mm <sup>2</sup>
Cable self-weight	46.12 N/m
Sag under self-weight at load point	29.276 m
Unstressed cable length of sections 1–2	125.88 m
Unstressed cable length of sections 2–3	186.85 m

Table 2 Verification example 1: Obtained displacements compared with published research

Researcher	Element type	Displacement	
		Horiz.	Vert.
O'Brien and Francis	Elastic catenary	-0.860	-5.627
Jayaraman and Knudson	Elastic straight	-0.845	-5.471
Jayaraman and Knudson	Elastic catenary	-0.859	-5.626
Tibert	Elastic catenary	-0.859	-5.626
Andreu <i>et al.</i>	Elastic catenary	-0.860	-5.626
Yang and Tsay	Elastic catenary	-0.859	-5.625
Huu and Seung	Elastic catenary	-0.859	-5.626
Thai and Kim	Elastic catenary	-0.859	-5.626
Vu <i>et al.</i>	Elastic catenary	-0.859	-5.626
Salehi Ahmad Abad <i>et al.</i>	Elastic catenary	-0.859	-5.626
Salehi Ahmad Abad <i>et al.</i>	Discrete elastic cat.	-0.855	-5.592
Coarita and Flores	Elastic catenary	-0.860	-5.627
Present work	Elastic catenary	-0.842	-5.684

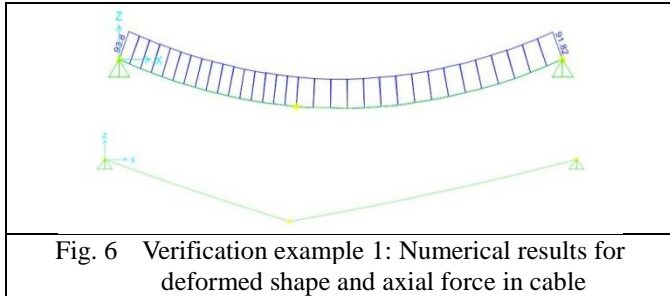


Fig. 6 Verification example 1: Numerical results for deformed shape and axial force in cable

4.2 Verification example 2: Single suspended cable

The studied example is a hyperbolic paraboloid cable net roof studied by Lewis (1984) and by Thai and Kim (2011). The spatial cable net consists of 4x4 quadrilaterals formed by 38 pre-tensioned cable segments spaced at 4m in x-direction and y-directions making grid with total dimension 24m and 16m. in plan, as shown in Fig. 7 (a). Cables in the x-direction with cross sectional area 350mm<sup>2</sup> have pre-tension force 90 kN, while cables in y-direction with cross section area 120mm<sup>2</sup> have 30 kN pre-tension force to maintain the structure initial geometry, and the cables modulus of elasticity is 160 kN/mm<sup>2</sup>. A vertical force of 6.8 kN force is applied at all internal nodes of the structure.

In this work, the finite element model for the cable net consists of 31 nodes and 38 cable elements, i.e. each cable segment is modeled as one cable element, and nonlinear analysis is carried out. The numerical results for cable net deformations and axial forces in cables are shown in Fig. 7. Table 3 shows the displacements at several nodes numerically evaluated in the present study compared to those obtained by Lewis (1984), Thai and Kim (2011) and Salehi Ahmad Abad *et al.* (2013). The displacements are very close to values obtained by the other researchers.

4.3 Case study: Existing cable-stayed roof structure

The studied application is the cable-stayed roof of the Conference Center located in Smart Village, Giza, Egypt, constructed in 2002 (Smart Villages 2015). The main structural system of conference center is a space truss roof supported by cables connected to six masts, as shown in Fig. 8. The space truss is supported along the perimeter on Steel columns connected to the space truss through steel plates that are welded and bolted.

A 3-dimensional model for the entire structure by finite elements is made in order to study the overall and detailed structural behavior under the studied loads. The properties assigned for the different materials of the structure and included in the computer program are as follows: Mild steel is used for all steel elements having yield stress  $f_y$  420 N/mm<sup>2</sup>; reinforced concrete elements have concrete cube compressive strength  $f_{cu}$  40 N/mm<sup>2</sup> and yield stress for cables is 350 N/mm<sup>2</sup>.

The finite element model consists of 1748 nodes, 46 cable elements, 6084 frame elements (used to model truss elements, purlins, concrete columns, steel columns and steel masts) and 1882 shells elements (used to model the structure external cladding). Fixed supports are assigned to the masts and columns and hinged supports are assigned to the cables. The three-dimensional finite element model for the conference center model is shown in Fig. 9 with its different structural elements such as cables, frames and shell elements.

The loads acting on the structure such as the roof loads, wind loads and seismic loads adopted in this study are given in Table 4. Dead loads are the weight of the roof cladding, purlins and truss elements, in addition to the weight of other utilities such as lighting fixtures, AC ducts and others. The live load on the inaccessible roof is assigned a value of 60 kg/m<sup>2</sup>. Wind load is considered as equivalent static lateral load. Seismic loads are calculated as horizontal and vertical loads, considering ground acceleration  $a_g/g$  equal to 0.15, as the

Table 3 Verification example 2 – Resulting displacements compared with published research

Researcher	Lewis (1984)			Thai and Kim (2011)			Salehi Ahmad Abad <i>et al.</i> (2013)			Present study			
Node	z-coord.	dx	dy	dz	dx	dy	dz	dx	dy	dz	dx	dy	dz
7	819.5	-5.14	0.42	30.41	-5.03	0.41	29.86	-5.05	0.40	29.55	-5.32	0.43	-31.21
8	1409.6	-2.26	0.47	17.70	-2.23	0.46	17.29	-2.23	0.40	17.16	-2.37	-0.48	-18.35
9	1676.9	0	-2.27	-3.62	0	-2.31	-3.61	0	-2.36	-3.19	0	2.37	3.02
14	687.0	-4.98	0	43.49	-4.92	0	42.85	-4.93	0	42.94	-5.13	0	-44.47
15	1147.8	-2.55	0	44.47	-2.55	0	44.26	-2.55	0	44.34	-2.65	0	-45.71
16	1317.6	0	0	41.65	0	0	42.08	0	0	42.14	0	0	-43.27

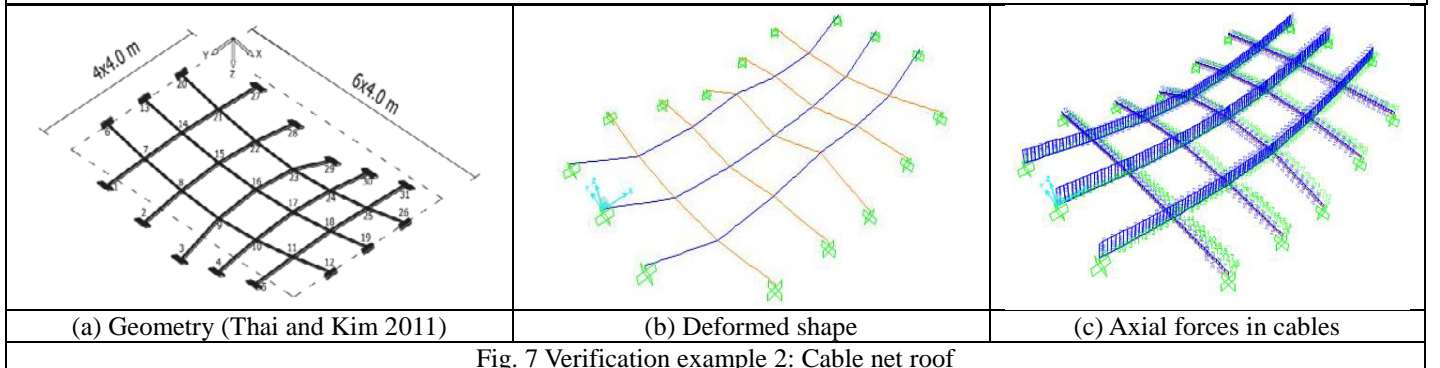


Fig. 7 Verification example 2: Cable net roof



Fig. 8 External view of the conference center (Smart Villages 2015)

structure is located is located in the third zone. The type of seismic design category is type C, spectrum type is type 1 and importance factor is 1.25 as it is a public structure. Nonlinear analysis is made for the structure under all load combinations as specified by AISC 360-10 (2010). The numerical results show that all stresses and deflections are in the acceptable range. The design safety level for all the structural members is shown in Fig. 9.

Design of the steel sections is made using LRFD design approach according to AISC 360-10 (2010). Optimization was carried out for the sections of the main truss only, so as to reach the least possible weight of the structure. The optimized weights are given in Table 5, compared to the corresponding weights of the existing structure. It is observed that the optimization procedure managed to decrease the structure total weight by about 16.88 ton.

### 5. Numerical analysis and optimization of different roof systems

In this numerical study, alternative solutions are proposed as roof structures to cover a column-free area with a square plan 48m×48m. Loading and design of sections are performed according to American code AISC360-10 Load and Resistance Factor Design (LRFD) [30]. Optimization is made to minimize the weight of structural elements and hence the total cost of the structure. The optimization procedure obtains minimum weight by selecting suitable sections from a standard set of steel sections. Stress constraints as well as maximum deflection constraints are imposed.

Table 5: Existing and optimized weights of structural members of the conference center

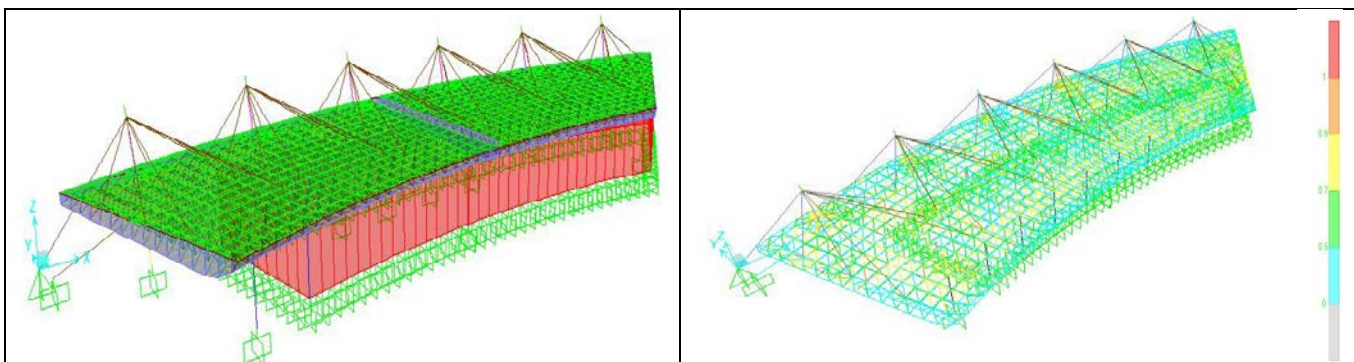
Structural members	Weight (ton)		
	Existing	Optimized	Reduction
Upper members	29.41	25.44	3.97
Diagonal members	71.70	63.53	8.16
Lower members	28.76	24.02	4.74
Cables	28.12	28.12	0
Masts	199.48	199.48	0
Concrete column	231.77	231.77	0
Corrugated Sheets	177.13	177.13	0
All structure	766.37	749.49	16.88

#### 5.1 Description of the studied structural systems

The proposed systems, shown in Fig. 10, are as:

a) *Plane truss system*: This system consists of plane trusses having 48m span and spaced at 6m, 7m, 8m, 9.6m and 12m, in order to choose the most suitable spacing. The truss upper, diagonal and lower members have hollow tube sections and it is supported on fixed columns at both ends. Horizontal and vertical wind bracing are provided. Purlins are placed on top and bottom of the roof.

b) *Cable-stayed plane truss system*: This system consists of 6 plane trusses with 48.0m span spaced at 8.0m and supported on fixed columns at the ends. The plane truss consists of upper, diagonal and lower members that have a different pipe sections. Horizontal and vertical wind bracing are present. Purlins are on



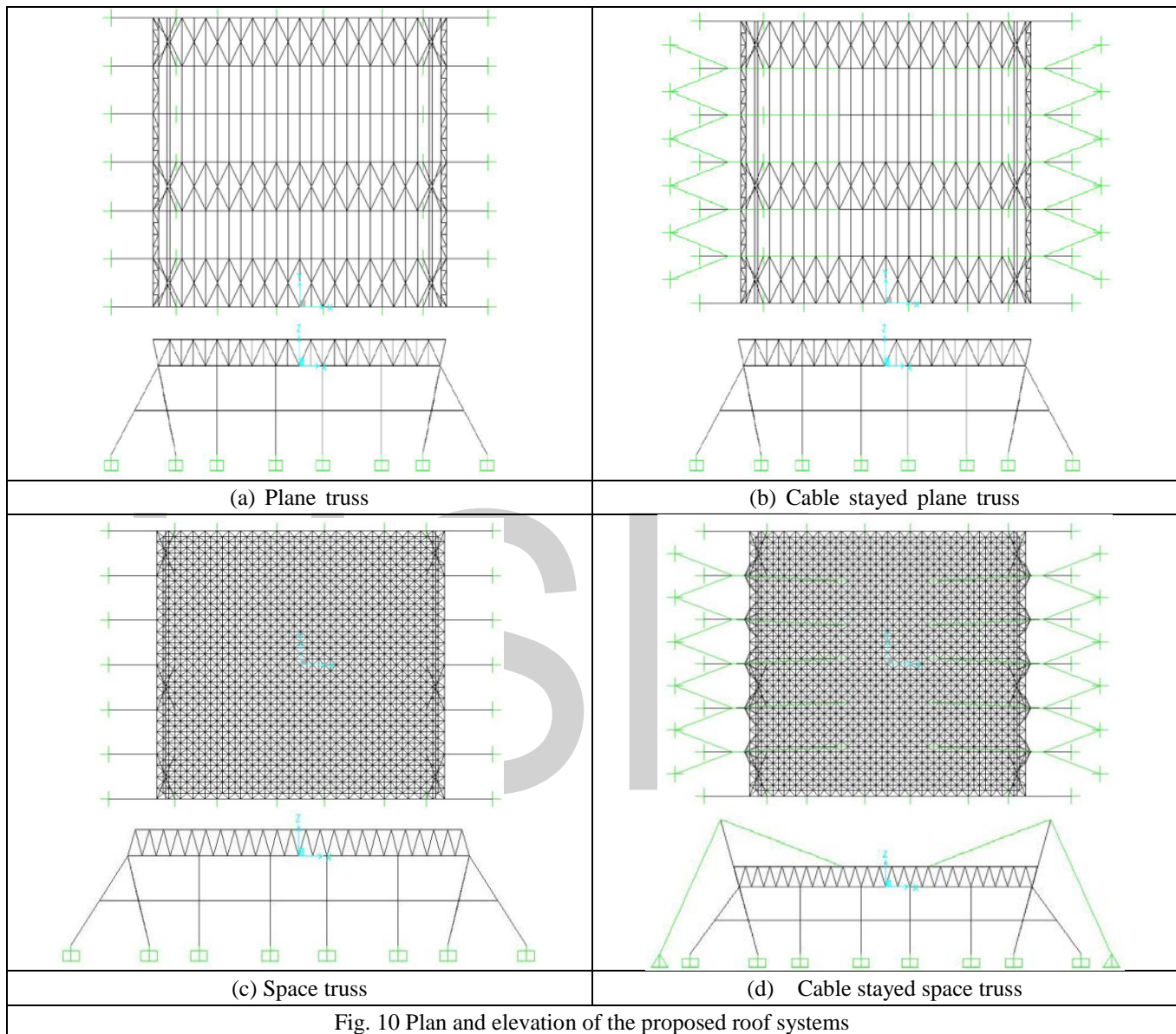
(a) Finite element 3D model for the whole structure

(b) Design safety level of the structural elements

Fig. 9 Finite element modeling and results for the conference center

top and bottom of the roof. Cables are used to connect masts to the roof truss at specified joints.

been used to model the truss members in the finite element model. For columns and masts, steel pipes were assigned with fixed joints at foundation level. Roof cladding was assigned as a



a) *Space truss system*: This system consists of a space truss with 48m span supported on fixed columns on the perimeter. Horizontal and vertical wind bracing resist wind load. Purlins are on top and bottom of the roof.

b) *Cable-stayed space truss system*: This system consists of a space truss roof covering a column-free square area of 48m×48m. The space truss is supported on concrete columns on the perimeter of the hall; in addition to cables connected to six masts on each side.

Plans and elevations for all the studied systems are shown in Fig. 10.

### 5.2 Finite element analysis

Finite element modeling is made for the four proposed structural systems using SAP2000 software. Steel pipes have

very light thin shell element as well as the wall cladding taking the distribution for all load types to transfer it into frames. All shell elements were divided at every points and frames. Steel cables were assigned in software program via input its diameter and its pre-tension force, for systems (b) and (d). The finite element mesh for all systems is shown in Fig. 11.

Analysis is performed for each structural system under all possible cases of loadings and load combinations of AISC360-10 code. The assigned loads including dead load, live load, seismic, wind and temperature loads of the proposed roofs are the same as that assigned in the conference center given in Table 4. The wind load on the structure was taken according to AISC360-10 [30]. Linear analysis is conducted for roof structures without cables and nonlinear analysis for roof structures with cables.

**5.3 Numerical results**

For the plane truss system, a numerical study is made where the spacing between the plane trusses is varied as 6m, 7m, 8m, 9.6m and 12m. Structural analysis is performed for each case while optimizing the design to reach the least possible weight. This study is made to reach the most suitable spacing which gives minimum weight for the whole structure.

The numerically determined maximum vertical displacements due to working load conditions were 0.1504, 0.0695, 0.0969 and 0.0515m for the plane truss, cable-stayed plane truss, space truss and cable-stayed space truss, respectively.

The allowable deflection is 1/300 of the span, which is equal to 160mm in both directions. The allowable limit was not exceeded by any of the studied systems. The deflection was highest for the plane

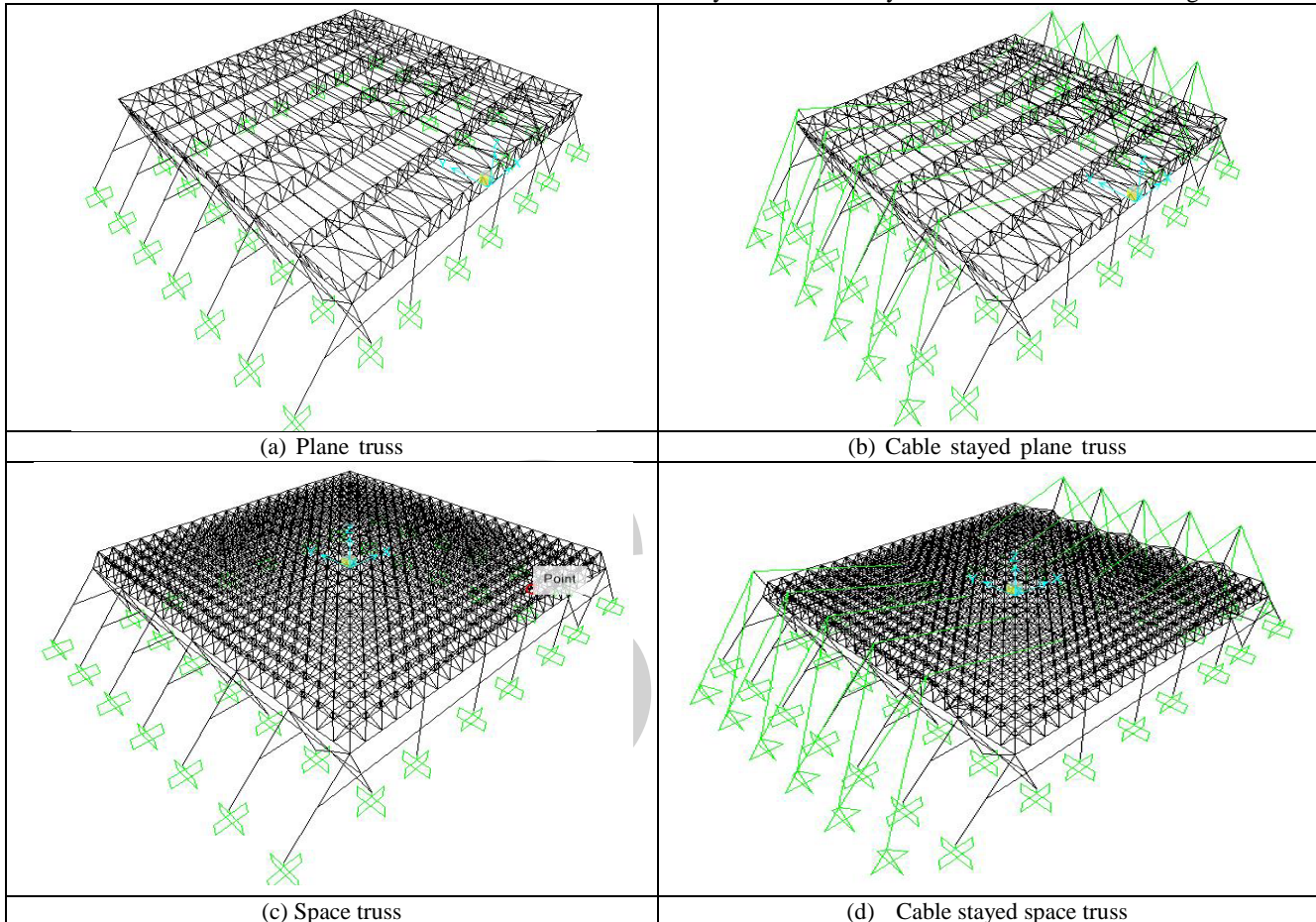


Fig. 11 Finite element 3 D mesh for the studied roof systems

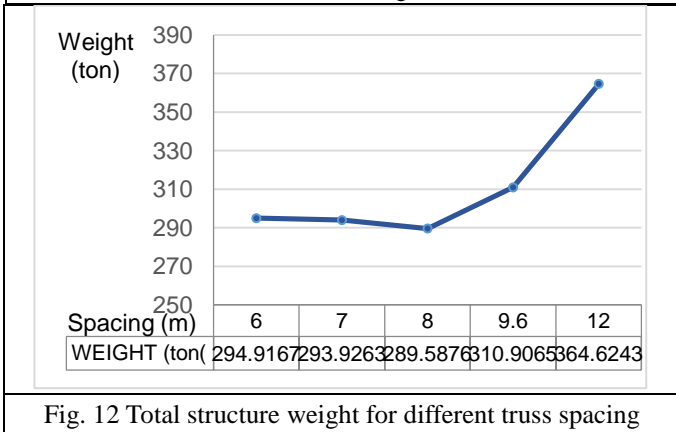


Fig. 12 Total structure weight for different truss spacing

The optimized weight of the whole structure for all cases is given in Fig. 12. It is concluded the economic truss spacing is 6 to 8m, with 8m spacing providing the least weight of the structure, and was adopted in the following studies.

truss system, approaching the threshold value allowed by design code, followed by the space truss. It is observed that the vertical deflection was significantly reduced to about 40-55% of its value by using cables to support the plane or space truss system. For the two cable-stayed systems, the lateral deformation of the masts was very small. For all the roof systems, the safety of design under ultimate load combinations is shown in Fig. 13.

For each structural system, optimization process was made to achieve minimum weight and consequently cost for the structure. For the space truss system, the obtained results after each trial are given in Table 6; the results are plotted for all the systems in Figs. 14 and 15. The final values of weight of structures and its members after optimization process to achieve minimum weight are listed in Table 7 and are compared in Fig. 16. Table 8 and Fig. 17 present the final comparison of the optimized weights of the different roof weight systems.



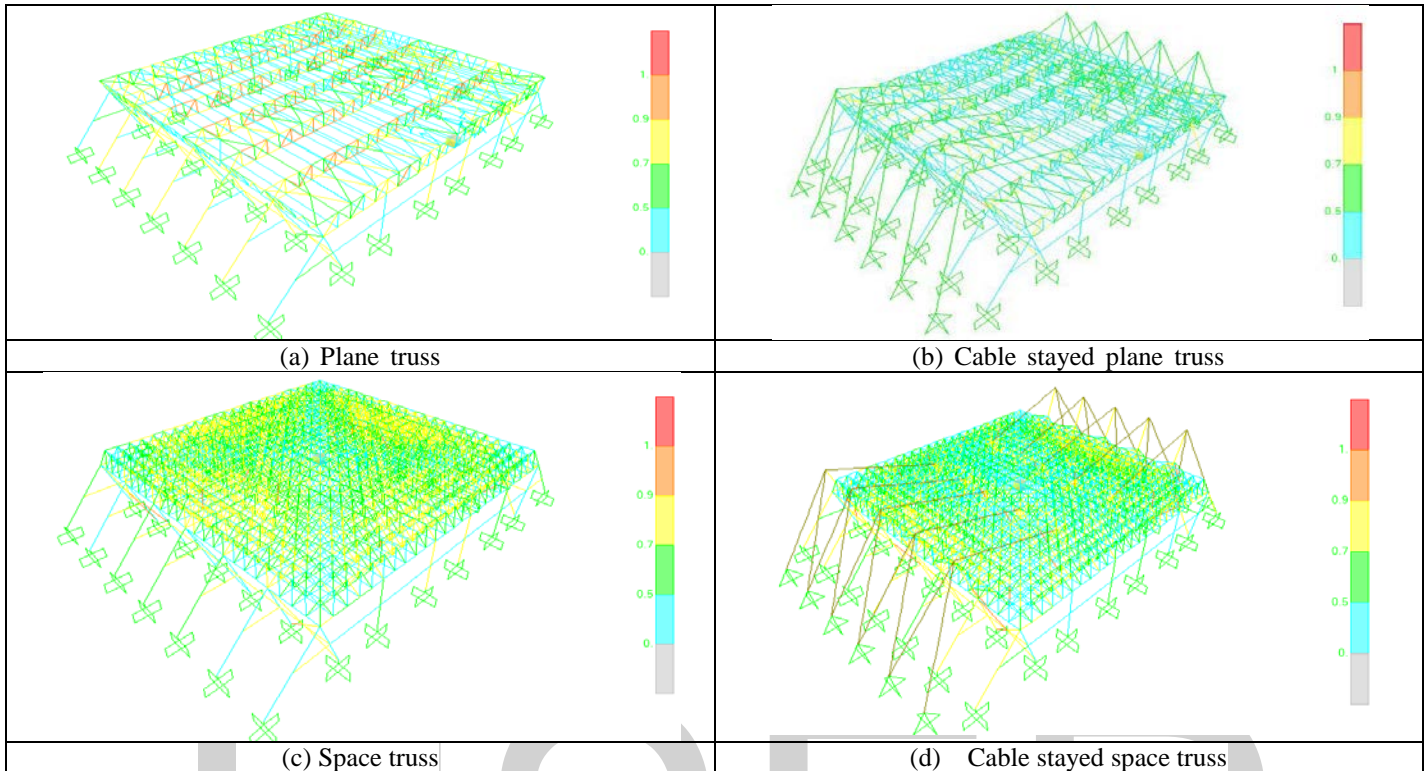


Fig. 13 Design safety according to all stresses for all systems

Table 6 Trials made to minimize weights of structural elements of space truss system

Optimization trial	Weight(ton)								
	Upper members	Diagonal members	Lower members	Main truss	Columns	Inclined columns	Outer beams	Vertical bracing	All structure
Opt1	39.691	137.364	43.143	220.198	14.351	58.383	8.503	6.178	308.997
Opt2	23.220	52.269	18.798	94.287	8.433	43.781	95.884	5.109	248.531
Opt3	15.244	41.121	10.448	66.813	8.433	41.665	95.820	5.142	218.835
Opt4	11.928	39.344	7.714	58.986	8.433	40.497	95.820	5.196	209.872
Opt5	10.265	38.689	6.488	55.442	8.433	40.497	95.820	5.196	206.317
Opt6	9.874	38.443	6.061	54.377	8.433	40.497	95.820	5.196	205.253
Opt7	9.763	38.262	6.035	54.060	8.433	40.497	95.820	5.196	204.935
Opt8	9.753	38.178	5.998	53.929	8.433	40.497	95.820	5.196	204.804
Opt9	9.753	38.156	6.001	53.909	8.433	40.497	95.820	5.196	204.785
Opt10	9.750	38.151	6.001	53.902	8.433	40.497	95.820	5.196	204.777
Opt11	9.750	38.144	6.001	53.894	8.433	40.497	95.820	5.196	204.767
Opt12	9.743	38.143	6.001	53.887	8.433	40.497	95.820	5.196	204.762
Opt13	9.743	38.143	6.001	53.887	8.433	40.497	95.820	5.196	204.762

Table 7 Optimization trials for main truss and total structure weights of all systems

Optimization trial	Plane truss		Cable- plane truss		Space truss		Cable- space truss	
	Main truss	All structure	Main truss	All structure	Main truss	All structure	Main truss	All structure
Opt1	25.412	301.099	25.412	301.099	220.198	308.997	265.239	367.286
Opt2	88.115	262.897	50.077	221.596	94.287	248.531	93.625	278.124
Opt3	58.367	304.653	27.108	237.960	66.813	218.835	61.469	236.211
Opt4	54.486	295.420	24.529	219.495	58.986	209.872	56.594	231.116
Opt5	53.474	291.801	23.255	214.301	55.442	206.317	55.706	230.249
Opt6	53.420	291.121	22.841	213.031	54.377	205.252	55.528	230.064
Opt7	54.118	291.541	22.665	212.650	54.060	204.935	55.504	230.071
Opt8	54.284	290.976	22.617	212.524	53.929	204.804	55.493	230.060
Opt9	54.528	290.167	22.606	212.513	53.909	204.785	55.483	230.049
Opt10	54.749	289.448			53.902	204.777	55.483	230.049
Opt11	54.916	289.615			53.894	204.770		
Opt12	54.909	289.608			53.887	204.762		
Opt13	54.889	289.588			53.887	204.762		
% wt. red.	37.708	3.823	11.042	29.421	75.528	33.733	79.082	37.365

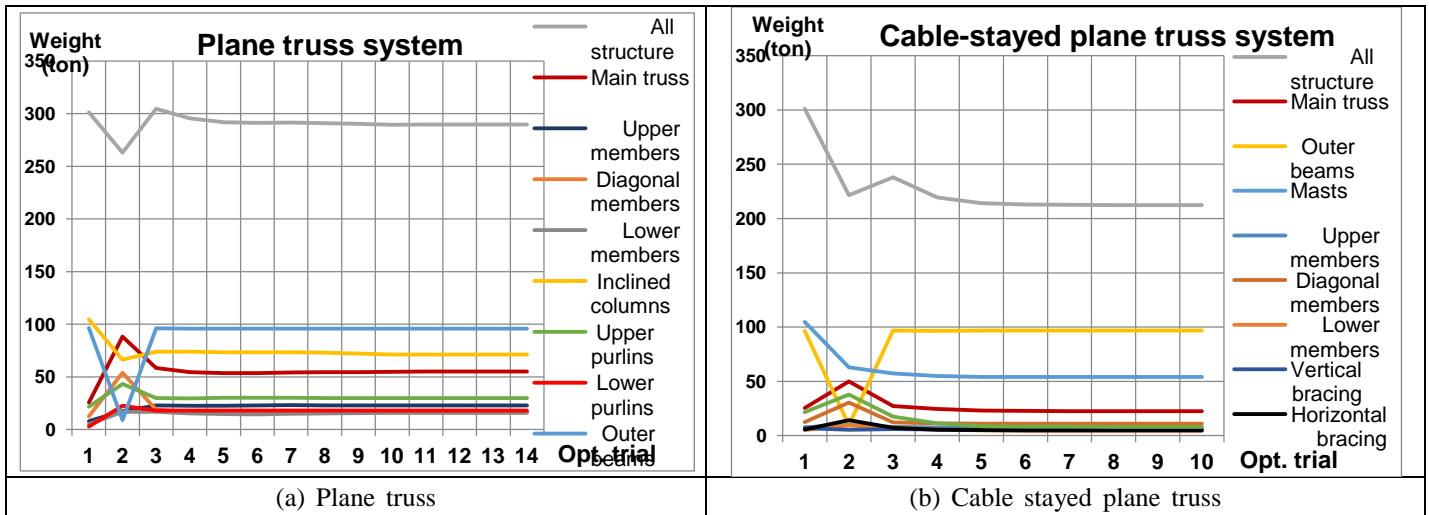


Fig. 14 Weight of structural elements of plane truss and cable-stayed plane truss systems through the optimizing trials

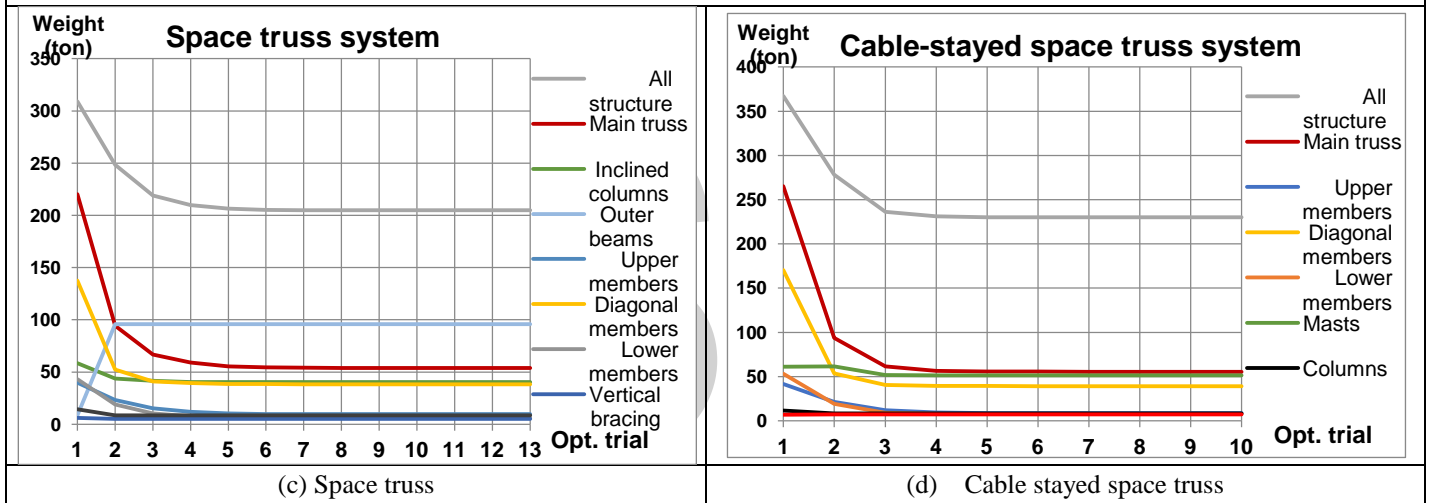


Fig. 15 Weight of structural elements of space truss and cable-stayed space truss systems through the optimizing trials

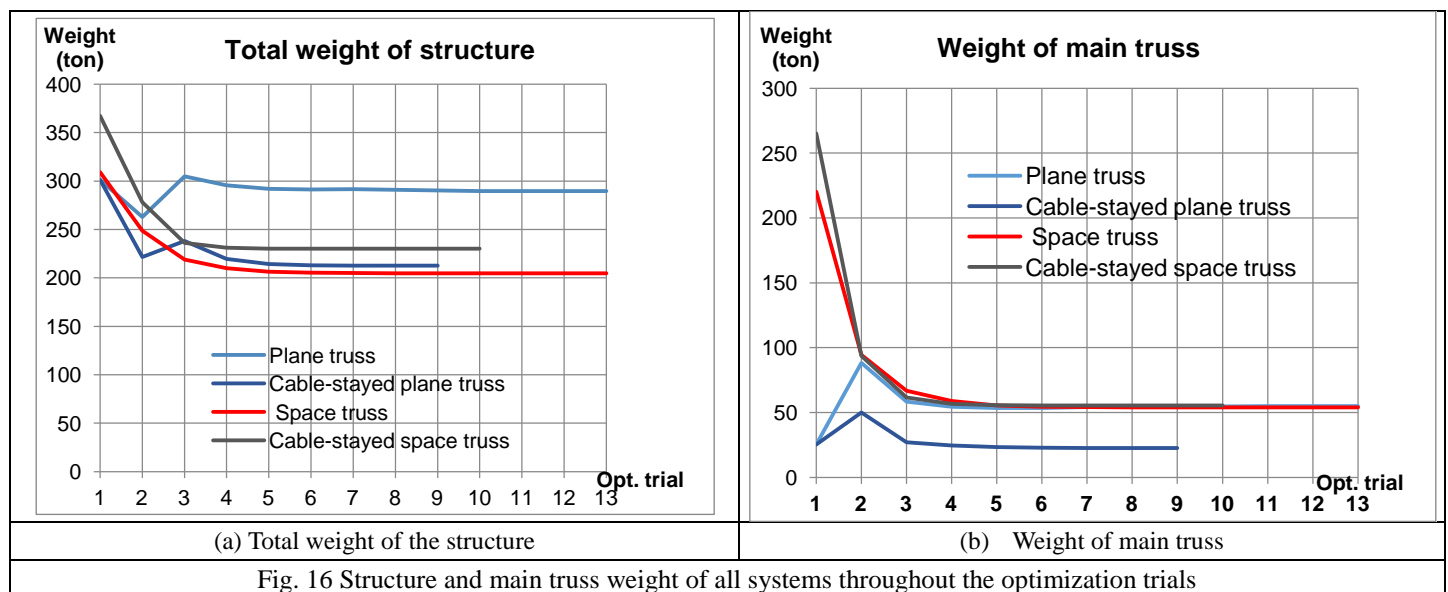


Fig. 16 Structure and main truss weight of all systems throughout the optimization trials

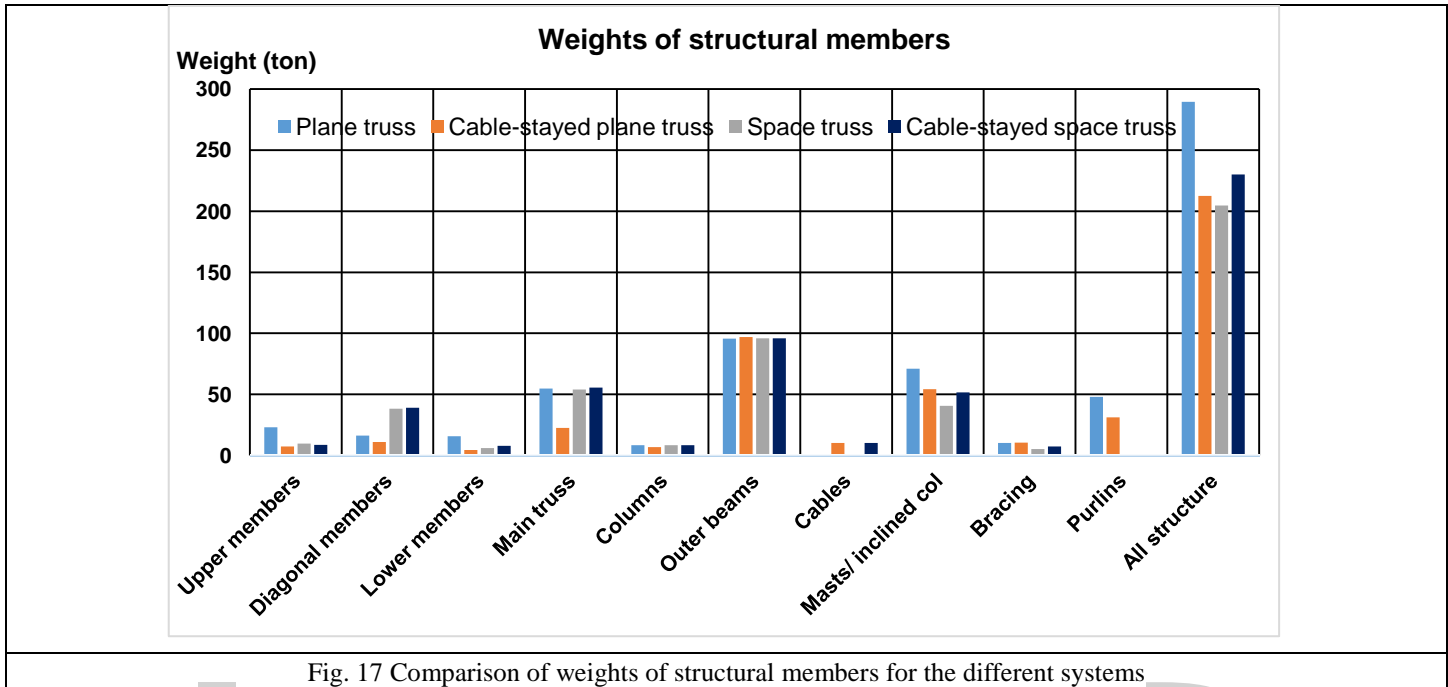


Fig. 17 Comparison of weights of structural members for the different systems

Table 8 Weights of structural members for all systems

Structural system	Upper members	Diagonal members	Lower members	Main truss
Upper members	22.913	7.252	9.743	8.595
Diagonal members	16.157	10.903	38.143	39.149
Lower members	15.818	4.451	6.001	7.738
Main truss	54.889	22.606	53.887	55.483
Columns	8.433	6.843	8.433	8.433
Outer beams	95.751	96.869	95.822	95.858
Cables	---	10.122	---	10.271
Masts/ inclined col.	71.031	54.128	40.497	51.503
Bracing	10.330	10.381	5.196	7.266
Purlins	48.021	31.142	---	---
All structure	289.588	212.513	204.763	230.049

Comparing the weight of the structure and its main parts, it is observed that the plane truss system has the highest weight, followed by the system of cable-stayed plane truss then the cable-stayed space truss; the space truss system had the least weight (having 70% the weight of the plane truss system).

Supporting the plane trusses by cables caused reduction in the total weight by 26.6%, while switching to space truss achieved about 30% and cable-supported space truss achieved 21% reduction in total structure weight. The space truss system is therefore considered the preferred system due to its simplicity in construction, small deflection value and lightweight

## 5. Conclusions

This paper presented finite element modeling and nonlinear analysis for the cable-stayed roofs of wide span halls. Formulation of the cable element is given together with the nonlinear analysis parameters and adopted optimization procedure. Application was made on examples found in

published research in addition to the roof of an existing conference hall. Finite element modeling and static nonlinear analysis were made using SAP2000 software under all possible cases of loading in order to study the structural behavior, check the safety of the structure and optimize the design of structural members to minimize weight. Further, a numerical study was made where four systems were proposed as roof for a square plan of side 48 meters. The proposed systems are plane truss, space truss, cable-supported plane truss and cable-supported space truss. Nonlinear analysis and optimization is carried out for each system in order to reach the least possible weight and cost. Comparison was made between the studied systems. The main conclusions obtained may be summarized in the following points.

- The results obtained using the proposed methodology show agreement with the results reported in published research work.
  - The adopted procedure was applied on the cable-stayed roof of an existing hall to investigate the response and safety of the structure under all loading conditions. Additionally, applying the proposed optimum design method managed to reduce the total weight of the structure by about 3.0%.
  - Lateral loads such as wind loads and seismic loads affect critically the structural behavior, as cable supported structures are usually light structures so the these types of structures should be designed primarily to withstand the lateral loads.
- For the suggested roof systems to cover a 48.0m×48.0m column-free area, the plane truss system provided the highest overall structure weight (and the largest mid span deflection), followed by cable-stayed plane truss then cable-stayed space truss. The least possible weight for the whole structure was accomplished through the space truss system, and the smallest deflection was provided by the cable-stayed space truss system.
- For plane truss system, the spacing between trusses of

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8.0m was found to yield the minimum total structure weight and thereby cost.

- The space truss system has the additional advantages of simple design, safety, economical cost, easy construction and easy conservation, over the cable-stayed plane or space truss systems.

- The conclusions derived from this study may not be identical for halls with larger spans, where further investigation will be required to reach such conclusions.

- In this study, no specific modeling was adopted to represent the connections, and thus further studies should take into account the effect of connections on the structural response.

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