ANALYSIS OF TENSION STRUCTURE

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CERTIFICATE

This is to certify that the Major Project entitled "**Analysis of Tension Structure**" submitted by **Mr. Trivedi Milind J. (03MCL17)**, towards the partial fulfillment of the requirements for the award of degree of **Master of Technology (CIVIL)** in field of **Computer Aided Structural Analysis and Design (CASAD)** of Nirma University of Science and Technology is the record of work carried out by him under my supervision and guidance. The work submitted has in my opinion reached a level required for being accepted for examination. The results embodied in this dissertation, to the best of my knowledge have not been submitted to any other university or institution for award of any degree or diploma.

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ABSTRACT

Cable supported structures have inspired and fascinated people for many decades. During the past two decades a number of large roofs have been built in the forms of cable-supported in which the main load-carrying elements are steel cables. These are subjected to tensile forces only, thus there are no stability problems and it is therefore natural to use high tensile steel rods or high tensile steel cables. Today, cable structures are recognized as unique and innovative structural solutions that create new and dramatic forms, while efficiently enclosing large volume spaces and providing new opportunities for transparency and natural light.

While cable-supported structures are not new, the three-dimensional configuration of this structural system with multiple levels of forestay cables that splay and support loads in three opposing directions makes this structure the first of its kind. The structural system for this problem is a masted cable-stayed roof system of masts that are sloped and stabilized by 15 cables (nine fore-stay cables and six back-stay cables) on both sides which in turn support both side inclined roof girders.

The behaviour of the Cable-supported roof is highly non-linear due to the presence of pretensioned cables. The non-linearity is due to the geometry and material. The large displacements due to the heavy loads are responsible for the geometric non-linearity and the stress-strain behaviour of the prestensioned cables is responsible for the material non-linearity. In this work only the geometric non-linearity is considered.

STAAD.Pro, a stiffness based software has been used to do the linear static and geometric non-linear static analysis. The results of linear and non-linear static analysis, displacements from STAAD.Pro, forces in masts girders and cables are compared.

The design guidelines are given for the whole sequence of works and finally concluding remarks are made at the end of the work.

Chapter 1 includes the introductory part of thesis, the classification of tension structures with different forms, phase of beahviour during their deployment, prestressing and service phase and objective of study.

Chapter 2 describes the literature review based on cable-supported roof in which linear and nonlinear analysis published by different authors.

Chapter 3 includes geometric configurations of cable mast, girders and anchorages. It also includes types of cable and their structural properties. Also includes fundamental criteria for the structure design.

Chapter 4 includes core theory of non-linear analysis. It also includes different types of non-linear solution procedures like incremental method, Newton Raphson method or Modified Newton Raphson method and mixed methods.

Chapter 5 includes 3-D modeling tips for cable-supported roof in STAAD.Pro 2003. It also includes how to done modeling of cable in STAAD.Pro. It also includes how to perform cable analysis static linear as well as static non-linear.

Chapter 6 includes different types of load and load combination for the analysis of cable-supported roof. It includes dead load, live load wind load with gust factor and dynamic analysis of earthquake forces.

Chapter 7 includes the results obtained from STAAD.Pro of static linear analysis. Also includes mode shape of the frame.

Chapter 8 includes the results obtained from STAAD.Pro of static non-linear analysis. And compare force and deflection of static linear and static nonlinear results.

Chapter 9 includes basic design of cable, main girder and mast of the cable supported roof.

Chapter 10 includes the summary, conclusion and further scope of work.

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NOMENCLATURE

$[K]$ = Global stiffness matrix of the structure

$\{D\}$ = Vector of joint displacements

$\{P\}$ = Vector of applied joint loads

K_0 = Elastic stiffness matrix

K_1 = Geometric stiffness matrix

K_{i-1} = Tangent stiffness matrix

Δ_0 = Initial deflection

n = No. load steps

R = Unbalanced load vector

E = Modulus of elasticity

f_{cb} = Elastic critical stress in bending

f_{bcx} perm. = Permissible bending compressive stress in extreme fibre about major axis

f_{bcx} calc. = Calculated bending compressive stress in extreme fibre about major axis

f_{bcy} perm. = Permissible bending compressive stress in extreme fibre about minor axis

f_{bcy} calc. = Calculated bending compressive stress in extreme fibre about minor axis

f_{ac} perm. = Permissible axial compressive stress in the member subjected to axial compressive load only

f_{ac} cal. = Calculated average axial compressive stress

Cm = A coefficient

f_{cc} = Elastic critical stress in compression

l/r_{yy} = Slenderness ratio in the plane of bending

CHAPTER 1 INTRODUCTION

1.1 GENERAL

There has been an increasing demand for increasingly larger span roofs during the past two decades. Tension structure is one of the options to construct the larger span roofs. Tension Structure are ones in which the main load-carrying members transmit applied loads to the foundation or other supporting structures by direct tensile stress without flexure or compression. Their cross-sectional dimensions and method of fabrication are such that their shear and flexural rigidities, as well as their buckling resistance, are negligible. There are two broad classes of tension structures: cable structures comprised of uniaxially stressed members, and membrane structures comprised of biaxially stressed members.

Cables have long been used in cable-supported bridges but only comparatively recently has their use as structural elements in roofs become popular. Due to their versatility, aesthetic appearance and ease of construction, the cable-supported structures have attained popularity in some advanced countries. They can provide large roofing areas without any intermediate supports such as for sport-stadiums, exhibitions halls, aircraft hangers, etc.

The general class of cable structures can be further divided into four subclasses:

1. Single cables in which single cable segments, or several simply connected segments, are subjected to loads predominantly in a single plane of action, e.g., suspension cables, tether or mooring lines, guy lines for towers or tents.
2. Cable trusses in which prestressed segments are multiply connected in a single plane and loaded in that same plane, e.g., cable-stayed bridges, double-layer cable-supported roofs.
3. Cable nets in which prestressed segments are multiply connected in a curved surface (synclastic or anticlastic) and loaded predominantly normal to that surface, e.g., hanging roofs, suspended nets.
4. Cable networks in which cable segments are multiply connected to form a three-dimensional frameworks, e.g., suspension networks, trawl nets, multiple-leg systems, simple paraboloid.

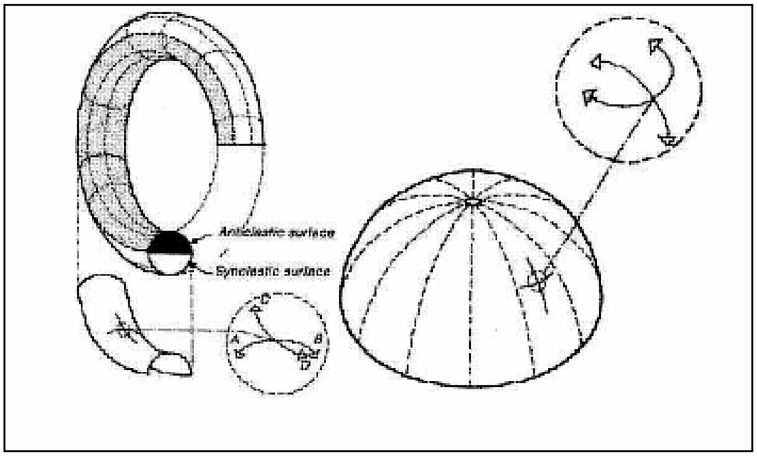


Fig. 1.1 Anticlastic Surface and Synclastic Surface

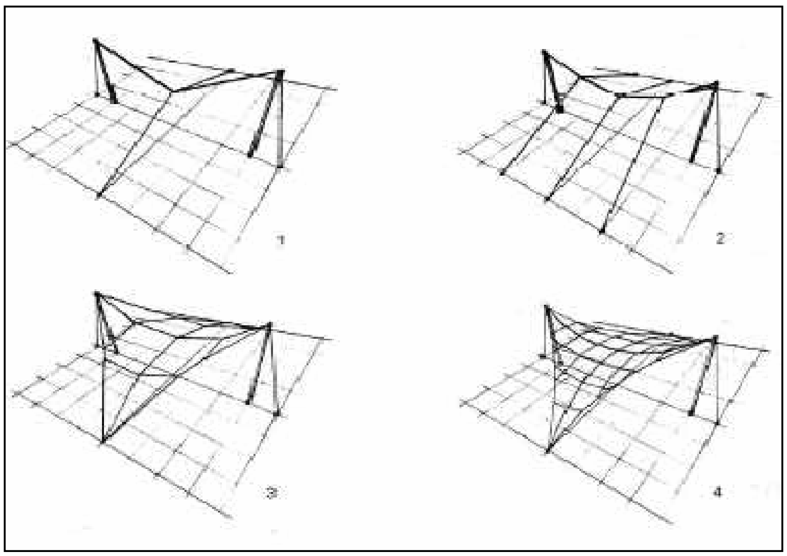


Fig. 1.2 Simple Paraboloid

There are four subclasses of membrane structures:

1. Air-supported structures in which an enclosing membrane is supported by a small differential air (or fluid) pressure, e.g., stadia roofs, inflated temporary shelters or storehouses.
2. Inflated structures in which highly pressurized tubes or dual-walled mats are used as structural members in a space structure, e.g., inflated beams, columns, or arches; dual-walled shells; air cushion roofs.
3. Prestressed membranes in which fabric or rubber like sheets are stretched over rigid frameworks and columns to form enclosures or diaphragms, e.g., tents, masted roofs.

4. Hybrid systems in which membrane panels span between primary load-carrying members such as prestressed cables and rigid members, e.g., reinforced fabric roofs, fluid storage tanks.

1.2 APPLICATION OF TENSION STRUCTURE

(1) They are lightweight and collapsible and therefore easy to transport and erect.
(2) They can be prefabricated in a factory, have low installation costs, and are potentially relocatable.
(3) For air-supported structures, the primary load-carrying mechanism is the habitable environment itself, i.e., a pressurized mixture of gases.
(4) The environmental loads are efficiently carried by direct stress without bending.
(5) They are load-adaptive in that the members change geometry to better accommodate changes in load patterns and magnitudes.

1.3 FORMS AND CLASSIFICATION OF CABLE ROOF

A cable roof can be defined as one in which a cable or a system of cables is used as a load-carrying structural element. Such roofs can be placed in different categories depending upon the criterion used for classification. In accordance with the manner in which cables are used, they can be classified as

(1) Cable-supported roofs
(2) Cable-suspended roofs
(3) Cable-cum-air-supported roofs

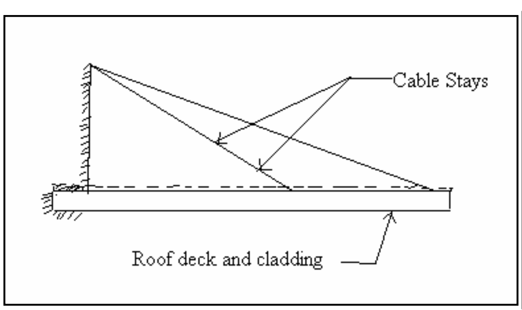


Fig. 1.3 Cable-supported Roof

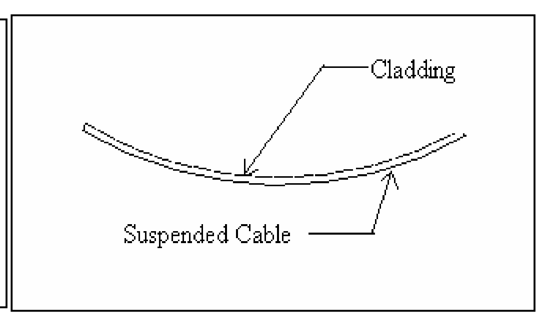


Fig. 1.4 Cable-suspended Roof

In cable-supported roofs, developed somewhat on the same lines as cable-supported bridges, the cables have only the secondary function of providing additional support for elements which are otherwise sufficiently strong to carry a major proportion of the loading. This type of roof has been built extensively in the United States and the European countries. In cable-suspended roofs, the system of cables carries the roof load directly and as such has a primary structural function. The cable system also serves as the false work for the erection of the cladding. Another class of cable roofs, which is of comparatively recent development and has now gained considerable popularity, is the air-supported roof. These roofs are mostly tent- or balloon-type structures which are supported by a combination of cables and air inflation. The use of air-supported roofs was most extensively illustrated in the 1970 International Exposition in Osaka, Japan.

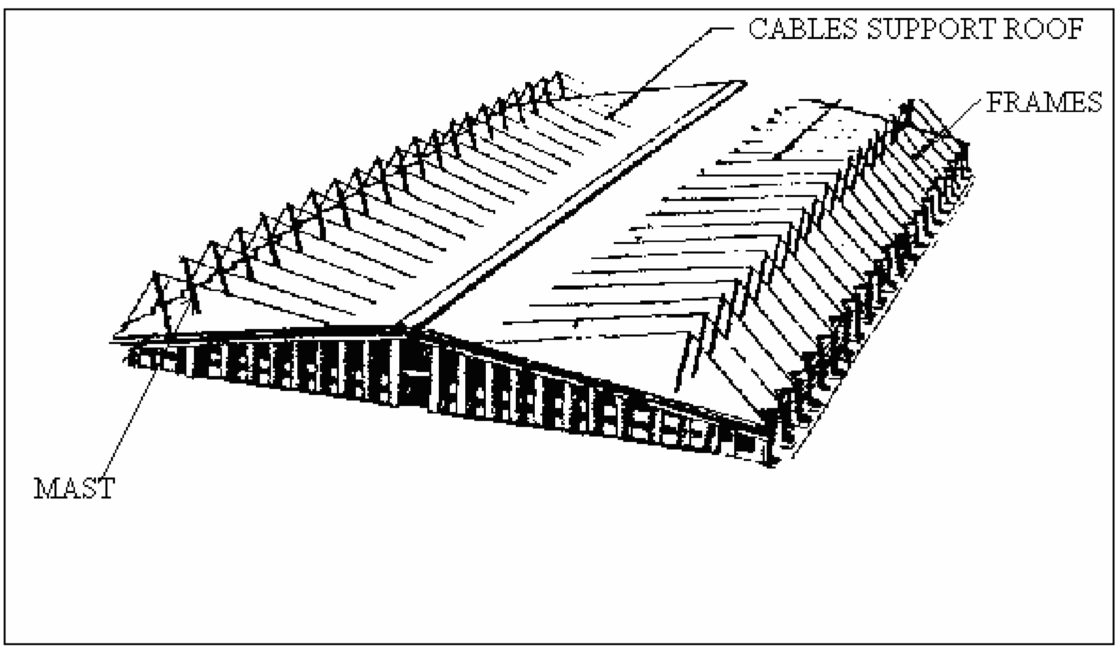


Fig 1.5 Cable-stayed Roof

1.4 PHASE OF BEHAVIOUR

The physical behavior of a tension structure during the application of loads can be divided into three primary phase. The first phase is deployment, in which the cable or membrane system unfolds from its compact configuration into a state of incipient straining. The second phase is prestressing, in which the cable or membrane systems deforms into a predominant equilibrium configuration under the action of dead weight, pressure, or other fixed lifetime loads. The final, or in-service, phase is the stage in which the fully prestressed system is subjected to variable live or dynamic loads during its service life.

1.4.1 Deployment Phase

In the deployment phase the cable or membrane unfolds with external forces counterbalanced by inertial forces only. During this dynamic process the structure is stress-free and the static and kinematics behavior is like that of a collection of rigid particles constrained by the topology of the fabricated cable or membrane segments.

The expansion from the compact configuration to the state of incipient straining is highly nonlinear, and no unique solution has been shown to exist. This need not be of concern in that stresses are negligible during this phase. However, one should take care to avoid "trapped" wrinkles or kinks which might lead to local tears or knots, high rates of deployment in relation to lack of stability under unexpected external loads, and transient overstress due to high initial velocities and accelerations when the structure is fully deployed.

1.4.2 Prestressing Phase

The second phase in the erection of a tension structure is prestressing, wherein the structure undergoes displacement from its state of incipient straining, hereafter referred to as the initial state, into a static equilibrium state dictated by the fabricated geometry of the structure and the prestressing forces. Since the displacements during this phase are large, this is a nonlinear problem, but unique solutions for the stresses and displacements are possible.

1.4.2.1 Geometry

In the stress analysis of tension structures reference geometry for the stress calculations must be defined. In most solution procedures one can easily specify the geometry for which all such calculations are made. However, this is not a trivial specification for tension structures. Stress calculations are desired for the prestressed state and that shape may be significantly different in dimension or configuration from the initial state.

Two alternatives are possible: (1) prescribe initial geometry, (2) pre-scribe prestressed geometry. The first alternative is more desirable from the fabrication point of view. This, however, leads to the more difficult problem of stress analysis in that it is necessary by incremental or iterative procedures to determine the nonlinear solution for the prestressed geometry and the stresses thereon. With the second alternative, a simpler procedure for stress analysis is possible. However, the initial state required to obtain the prescribed prestressed shape exactly may be impractical to fabricate.

1.4.2.2 Static

The prestressing phase is a static equilibrium problem in that the state of stress and shape due to a predominant static load is required. If the reference geometry is specified on the initial configuration, the equilibrium equations are nonlinear since the loads and stresses act on the prestressed configuration which has unknown locations, orientations, and curvatures of line and area segments. Because of the assumptions of negligible bending rigidity, transverse loads are balanced by gradients in curvature multiplied by the internal tensions. Thus iterative solutions are necessary to determine the stress state; assumed displacements lead to calculable stresses which lead, in turn, to new assumptions for displacements.

1.4.2.3 Kinematics

Because of the extreme flexibility of tension structures (no bending resistance, small cross sections), large displacements occur during the prestressing phase and nonlinear strain-displacement relations should be used. Strains will be small, but relative rotations are large and thus second-order terms of displacement gradients are significant, i.e. line segments may not change much in length, but they do translate and rotate appreciably due to transverse loads.

1.4.2.4 Material Behaviour

During the prestressing phase semirigid translations and rotations of differential segments predominate over strain effects. Thus, it is often sufficient for preliminary design purposes to assume inextensible behavior for most engineering materials unless the system is highly redundant. In such cases incremental procedure can be used and piecewise linear elastic behavior can be assumed. It is only in the last stages of prestressing and in the subsequent in-service phase that constitutive equations play an important role.

1.2.4.5 Constraints

Boundary conditions for the prestressing phase involve prescription of surface tractions due to external prestressing forces, of edge forces, and of support motions. Dead-weight loads on cables and membranes are conservative forces. If the initial shape is taken as the reference configuration, the external force terms in the equilibrium equation will be displacement-dependent. If the prestressed shape is used as the reference configuration, the load nonlinearity will instead occur in the equations used to determine the initial shape.

Tension structures have negligible bending and buckling resistance. The boundary restraints must be consistent with the behavior. Clamped edges cannot be realized; instead, the cables and membranes will undergo localized kinking.

1.4.3 In-service Phase

The third phase in the behavior of a tension structure is the in-service phase, wherein various static live loads, e.g., snow loads, and various dynamic loads, e.g., wind or wave loads, which are expected to occur during the service life of the structure, are superposed on the prestressed configuration. Depending on the relative magnitudes of those loads compared to the prestresses, one can consider the behavior in this phase linear or nonlinear. The prestressing stiffens the structure, and the additional deflections due to in-service loads are considerably smaller than the prestressing displacements.

The geometry of the prestressed configuration can be used as the basis for stress calculations during the in-service phase in that usually only slight perturbations of that shape occur. If large excursions under added load are expected, then an incremental loading technique can be used in which occasional recalculations of reference geometry and prestresses are made before additional increments of load are considered. This same procedure can be used to predict a new prestressed configuration if there is significant change in the predominant loads on the tension structure.

The equations of statics or equations of motion for the in-service phase are linearized equations in that a fixed geometry is considered and additional displacements are small. Accounting for the effect of prestressing on the membrane or cable stiffness, i.e., includes some nonlinear effects geometric stiffness contributions are considered. In dynamic analyses of tensile structures it has been found that the magnitude of prestress has an effect not only on the frequencies of vibration but also on the mode shapes.

The behavior during the in-service phase is not in general statically determinate. The kinematic and constitutive equations must be incorporated into the equations of motion. Piecewise linear material behavior is often assumed. An important aspect in the analysis of in-service behavior is the calculation of reductions in tensile stresses due to added loads. Slackening of cables and wrinkling of membranes should be avoided, or if unavoidable, their extent and effect on global stability of the structure predicted. Flutter and dynamic wrinkling of cable-supported and air supported roofs under wind loads have been shown to be important considerations in the design of such structures.

1.5 OBJECTIVE OF STUDY

The structural response of cable-supported roof to externally applied loads is always nonlinear because of the large displacements, which occur substantially changing the geometry of the structure. Due to lightweight it is more susceptible to wind internal suction and pressure.

Following are the objectives of the study:

- To develop geometry of the cable-supported roof in a general stiffness based program STAAD.Pro 2003 and perform a static linear analysis.
- Due to the geometric nonlinearity analyze the structure with same geometry and loads and perform static non-linear analysis.
- Compare the results of static linear and static non-linear analysis.

1.6 APPROXIMATION AND ASSUMPTION

In carrying out the analysis, certain assumptions are made to simplify the computational work.

(1) The cable is treated as being completely flexible. i.e. it can not sustain any bending moment.
(2) The cable is considered incapable of taking any compressive forces.
(3) The cladding does not contribute to the stiffness of the roof structure.
(4) The intersection of two or more cables is treated as a joint.
(5) The cable elements lie along straight lines between the joints. This implicitly assumes that all the loads including the self weight of the cable are acting at the nodes only and that the cable element itself is weightless.

1.7 PROBLEM FORMULATION

Data of the problem:

1) Longitudinal cable configuration : Radiating Harp Type
2) Span of the Structure : 60m
3) Plan Dimension : 60m x 100m
4) Height of Roof Level : 10m
5) Height of mast : 32.86 m @ 10° inclination
6) Spacing of Girders : 8.33m c/c
7) Spacing of Purlin : 2.15m
8) Spacing of mast : 25m c/c
9) Cables
   (a) 9 forestay cables and 6 backstay cables of 36mm diameter
   (b) Cables are anchored outwards at 7.62m height of mast at three levels
10) Tie Back Frame at 8.62m either side of the mast

Section Proprieties:

1)	Main Girder :	ISMB 600
2)	Purlin :	ISMB 300
3)	Mast :	ISHB 450
4)	Tie-Back Column :	ISHB 450
5)	Tie-Back Frame :	TUB 1501506.0
6)	Cross Bracing :	TUB 1501506.0
7)	Cables :	36 mm Dia

Material Properties:

1) Modulus of Elasticity of Steel : 2.05×10^5 N/mm^2
2) Modulus of Elasticity of Cable : 1.95×10^5 N/mm^2
3) Poisson Ratio : 0.3
4) Coefficient of Thermal Expansion for Steel : $1.2 \times 10^{-5}\ /^{o}C$
5) Coefficient of Thermal Expansion for Cable : $1.17 \times 10^{-5} /^{o}C$
6) Density of Steel : 76.8195 kN/m^3

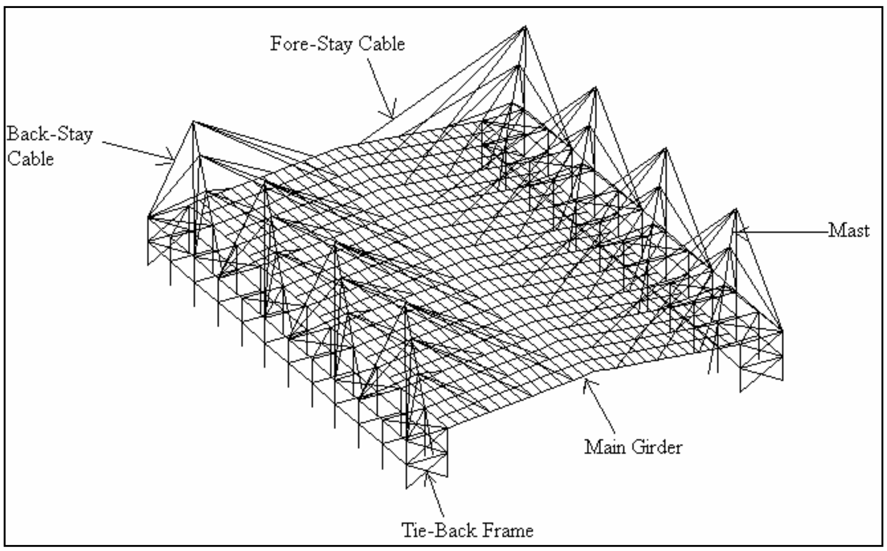


Fig. 1.6 3-D View of Structure

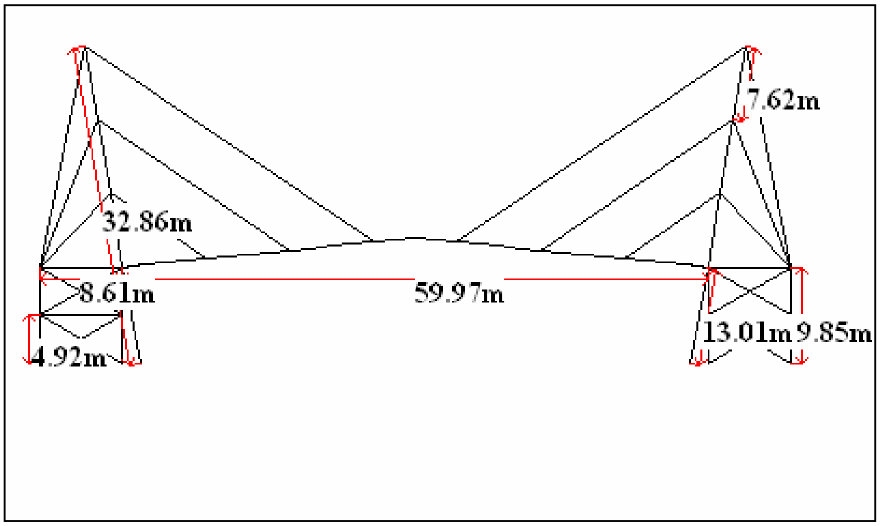


Fig. 1.7 Front View of Structure

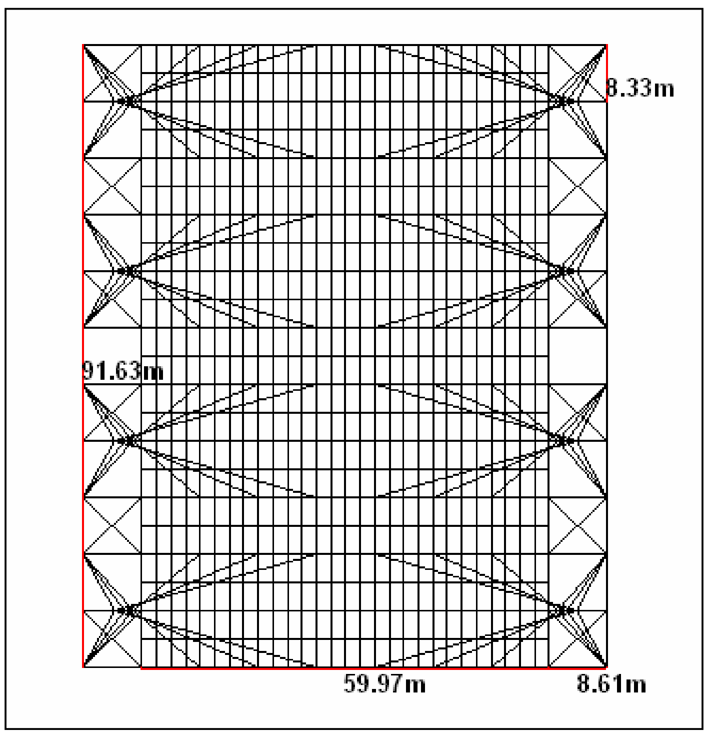


Fig. 1.8 Plan of Structure

2.1 GENERAL

Survey from various literatures such as research papers and different books has been carried out to support the present work. Literature survey has been carried out for the linear and nonlinear analysis of cable supported roof. Several research papers and books have been studied for 3-D analysis of cable supported roof for the different load cases and load combination.

2.2 CABLE-SUPPORTED ROOF

MAMORU KAWAGUCHI AND MASARU ABE (1) West Japan Exhibition and Sports Hall designed by the authors in cooperation with the architect A. Isozaki is covered by a group of cable stayed spatial roofs. In some aspects a cable stayed roof shares common structural properties with a cable-stayed bridge, but in some other aspects they possess quite different nature from each other.

Since this type of roof is rather new in the field of building structures, there are some fundamental criteria for its structural design, which need special considerations.

In the present paper outline of the structure is first described, and then a few fundamental criteria for its structural design is investigated.

Structural features of the West Japan Exhibition and Sports Hall designed by the authors have been described. Fundamental criteria necessary for design of a cable-stayed roof are considered. It has been found that there exists a relationship between the extensional rigidities of staying cables and the flexural rigidities of the roof girders, which gives useful information to find favorable stresses in them.

Stresses in the cables and the girders are also governed by the height of anchorage of the cables to the column. It has been found that the lateral stability of the column is largely assured by the restraint by the adjacent units, especially in a case such as the present design where the backstay cables are arranged in such a way that they have no components in lateral direction.

F OTTO AND FRIEDRICH-KARL SCHLEYER (2) has discussed basic concepts of tensile structure. In this reference has also discussed about cables, cable nets, and cable structures. General principles and classification of tensile structure has been discussed.

DAVID E ECKMANN (3) this paper presents several unique aspects of cable and cable-supported structures, and some of the unique design considerations. This paper also includes a case study on the University of Chicago Ratner Athletics Center, a recently completed cable-stayed structures that covers 150,000 square feet of health, fitness and sporting activity space. The projects features a first of its kind asymmetrically supported cable-stayed system that gracefully suspended S-shaped roof that float over the large volume spaces. This innovative structure utilizes 10-story tall composite masts and a series of splayed cables that support shallow curved steel roof members. The tapered masts are stabilized by back-stay cables anchored in place by massive concrete counterweights that counteract the weight of the roof.

R. E. SHAEFFER (4) has discussed basic information about the nature and characteristics of tensioned fabric structures. The basics of structural geometrics form and design methods like non-linear displacement, dynamic relaxation and force density method has been discussed.

2.3 NONLINEAR ANALYSIS

PREM KRISHNA (5) has discussed the theory and implementation of the procedures for the stiffness analysis, both static and nonlinear analysis in the reference. In the reference, a type of cables, different types of load and load combinations has also discussed.

J LEONARD (6) has explained the application of tension structure in this reference. Author also discussed about the static analysis of cable segments and systems and dynamics of cable systems.

A K JAIN (7) has discussed about geometric non-linear analysis used for the structure. In this reference, types of non-linearity, computer method of solutions for the non-linear problems and use of various techniques for improving the solution of the non-linear problems are discussed.

JOHN B. SCALZI AND WIILLIAM K. MCGRATH, FELLOWS, ASCE (8)

In this paper author has presented mechanical properties of structural cables for strands and ropes. They also published Load Vs Elongation graph which indicates that the slope of the line is established between the load points of not less that 10% of the minimum breaking strength and not more than 90% of the prestresching load.

The prestreching load applied to a cable does not usually exceed 55% of the rates minimum breaking strength of strand or 50% of the rated minimum breaking strength of rope and essentially eliminates the constructional stretch of cable.

NICHOLAS F MORRIS, M. ASCE (9) Flexible roofs, such as cable nets and air-supported structures, are a recent development in construction technology. Nonetheless, because these structures permit large areas to be spanned with an extremely small amount of material, their use has become widespread for both permanent and temporary structures. The basic design problem for such roofs is, of course, their response to wind, and, in fact, air supported structures have been severely damaged by wind. In recent years, computer programs have been developed for both static and dynamic analysis of these flexible roofs. Since their behavior basically nonlinear. A major difficulty with such analyses is the description of the action of wind on flexible roofs.

LOUIS F. GESCHWINDNER, JR. MASCE (10) Numerous methods for dealing with the general nonlinear dynamic analysis problem have been presented in the literature. Implicit approaches have been proposed by Newmark, Houbolt, Park and others.

JUNG-HWI NOHL, J0NG-HEON PARK, SUNG-HAN CHO AND KYU-Il CHO (11)

The primary objective of initial shape analysis of a cable stayed bridge is to calculate initial installation cable tension forces and to evaluate fabrication camber of main span and pylon providing the final longitudinal profile of the bridge at the end of construction. In addition, the initial cable forces depending on the alternation of the bridge’s shape can be obtained from the analysis, and will be used to provide construction safety during construction.

In this research, conducted numerical experiments for initial shape of Ko-ha bridge, which will be constructed in the near future, using three different typical methods such as continuous beam method, linear truss method, and IIMF (Introducing Initial Member Force) method.

2.4 CODAL PROVISION

A.S.C.E. 19-96 (12) has been dealing with the structural application of steel cables for building. As there is no Indian code detailing in the cable-stayed structure, this ASCE standard also states the minimum breaking strength of cables shall always be at least twice the maximum cable design loads, including the envelope of loading combinations of cable selfweight, structure dead load, cable prestress forces, and live load and environmental load combinations. Cables should also maintain a *minimum* tensile force under all loading conditions to minimize visible cable sag and potential for induced cable vibrations. Maintaining minimum cable tensions is also critical in achieving the required stiffness necessary to stabilize the axial compressive masts and other components of the structure.

I. S. 800 (13) gives procedure of calculation of wind load using gust effective approach for cables, mast and roof.

I.S. 1893 (Part 1): 2002 (14) gives procedure for dynamic analysis of structure using response spectra method.

CHAPTER 3 GEOMETRICAL CONFIGURATION AND FUNDAMENTAL CRITERIA FOR STRUCTURE DESIGN

3.1 GENERAL

The Successful application of cable-supported structure was realized only recently, with the introduction of high strength steel, orthotropic type decks, development of welding techniques and progress in structural analysis. The development and application of fast processor computers opened up new and practically unlimited possibilities for the exact solution of these highly statically indeterminate systems and for precise dynamic analysis of their 3-dimensional performance.

3.2 GEOMETRICAL CONFIGURATION

The cable-supported roof can have a large variety of geometrical configurations; limited only by the creativity of the designer. The layout of the cable-supported, the style of the columns and the type of the roof can be easily adjusted to suit the engineering requirements and to enhance the architectural beauty.

The various components of cable-supported roof are;

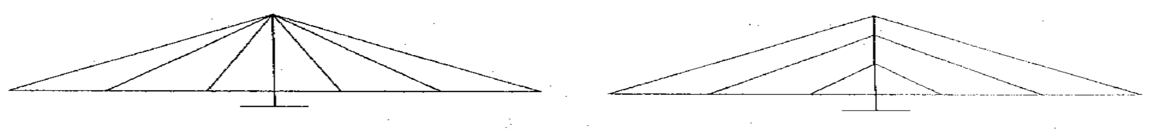
- Cable system supporting the roof
- Columns or Tower
- Girder
- Anchorage

3.2.1 Cable System Supporting the Roof

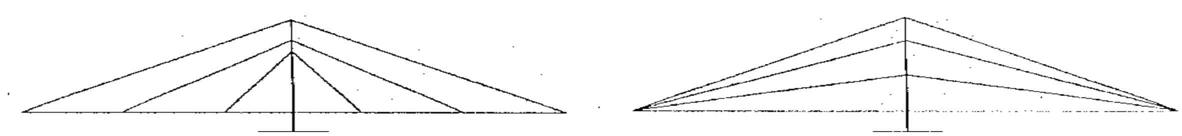
The arrangement of cable-supported is having crucial importance in the design of cable-supported roofs because of its strong influence on the overall structural performance. Furthermore, the spacing of the cables on the roof determines the type of construction procedure to be used. For example, for a cost-effective cantilever construction sequence in which temporary stays are not used, the spacing of the cable at the roof should not exceed the length of the roof segments. Also to facilitate the removal of a damaged or corroded cable, the cable spacing at the roof should be such that a single cable may be removed and replaced without using the false work and the system should not be overstressed.

Longitudinal Cable Arrangement

According to various longitudinal cable arrangements, cable-supported roofs could be divided in to the following four basic systems as shown in Fig. 3.1



(a) Radiating System (b) Harp System



(a) Fan System (b) Star System

Fig. 3.1 Longitudinal Cable Arrangements

1) **Radiating System**

In this system, all cables are concentrating to the top of the column. Structurally, this is the most efficient system, as by taking all cable to the column top the maximum inclination to the horizontal is achieved and hence it increases the load-balancing component of the cable tension, at the same time decreases the axial force in the roof. The primary force in the column is the axial compression; the bending of the column is minimized. However, where a number of cables are taken to the top of the column, the cable supports within the column may be very congested and highly stressed zone, so the construction and the detailing becomes very complex in that region.

2) Harp System

In this system, the cables are placed parallel to each other and hence the cable-anchors are distributed uniformly both along the height of the column and along the roof deck. This simplifies the detailing of the anchors at the both ends. It also results in ease of installation of cable-supported during the construction and at the time of any future replacement of damaged-stays.

Although from structural point of view, the harp pattern is least efficient. From an aesthetic point of view it is the most elegant solution because of the harmonious and uncluttered appearance of cables running parallel to each other. However, it causes large bending moments in the column.

3) Fan System

The fan system is a combination of harp and radiating patterns, avoiding individual shortcomings of both the systems. By spreading the cable-supported in upper part of the column, anchorage details can be simplified. Whereas in the lower part of column, the stays can be more steeply inclined resulting in a more efficient load carrying mechanism.

4) Star System

The star system is an aesthetically attractive cable arrangement. It contradicts the principle that the points of attachment of cables should be distributed as much as possible along the main girder. This system is not sound structurally, and hence it has been obsolete.

3.2.2 Supporting Mast System

The masts are the principle compression members transmitting the load to the foundation. Columns are of different types to accommodate different cable arrangements, roof site conditions, design features, aesthetics and economical considerations. Generally the arrangement of the cables determines the design of both column and roof deck.

Leonhardt recommends that columns should be slender in the longitudinal direction so that unbalanced horizontal cable components caused by any live loads in the central spans are transmitted to the ground through the backstays rather than through the bending of the columns.

The height of the column is determined from several considerations such as cable arrangement, visual appearance, economics etc. The recommended height of the column is 0.2 multiplied by span of the length for column below roof and 0.3 multiplied by span of the length for column above roof.

3.2.3 Girder Cross-Section

The deck girder is designed to be as slender as possible without comprising its safety against geometric stability. Other design considerations include the aerodynamic behavior and ease of cable anchorage. The shape the cross-section depends on the type of cable system used.

Generally in cable-supported roof girders are used to be of steel section. It is made of steel section of either I section or now days tubular section are very easily available in the market.

3.3 CABLE TYPES AND THEIR PROPERTIES

The cables used in cable-supported roof, at present fall in the following categories:

- Parallel-bar-cables
- Parallel-wire cables
- Stranded cables
- Locked-coil cables

The choice of a particular cable depends upon the need of the mechanical properties, i.e., modulus of elasticity, ultimate tensile strength, fatigue, durability etc. (Table 3.1), and the structural and economic criteria, i.e., erection, design of anchorages etc.

In the fabrication of parallel-bar stays, the bars must be couple together which lowers the fatigue resistance that that of a wire strand. Parallel-bar stays have only been used in two cable-stayed bridges during 1971 and 1985.

The higher modulus of the parallel-wire is the result of the combination of 50-350 wires being placed in a parallel position, and therefore approaching the elastic characteristics of the individual wires. Fatigue strength is satisfactory mainly because of their good material properties.

A rather recent innovation for cable-stays is the use of parallel 15 mm diameter 7- wire prestressing strand. These strands have a relatively high breaking strength, which result in a less volume and less weight of steel.

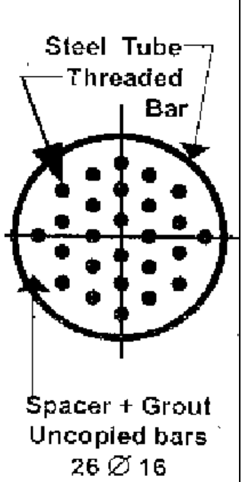
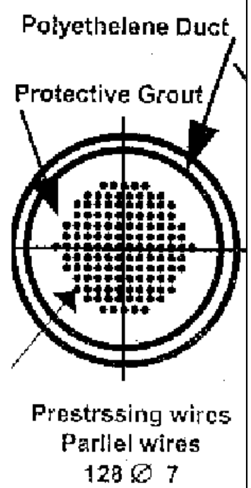
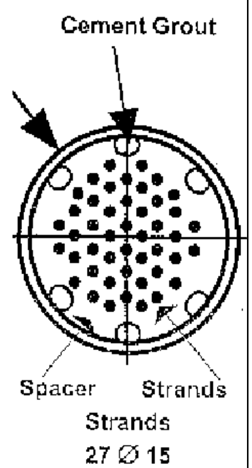
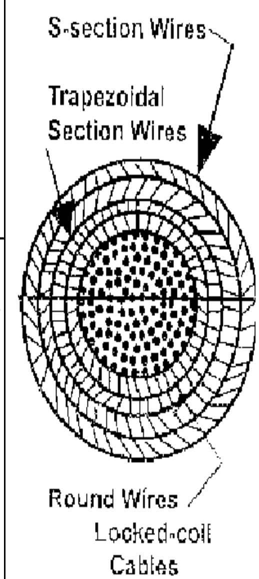
The corrosion resistance of locked-coil strand is increased by virtue of exterior tightly locked, Z- or S-shaped wires. The modulus of elasticity is also high. The advantages of lock-coiled cables in the case of placing are:

(1) The economy arising from the fact that ducts and grouting are not necessary,
(2) The reduced anchorage spaces.
(3) The great flexibility that makes it possible to use guiding saddles at the columns instead of intermediate anchorage.

The cable of the cable-supported roofs is more prone to corrosion in comparison to cable suspended roofs mainly because they are stressed up to 50% of ultimate tensile strength as compared to 35% in suspension roof. The most common method for protection against corrosion is galvanization of the wires. An alternative to galvanized the wires is to coat them with CRAPAL, an alloy of zinc with 5% aluminium. Flexible high-density Polyethylene (HDPE) pipe around the bunch of wires and grout the gap between them with cement mortar is the best solution for corrosion protection.

The design of the stay-cables is governed by fatigue strength of the cables, in the case of low side span to main span ratio and high live load to dead load ratio. Backstay cables of a cable supported structure experience most severe fatigue. Permissible maximum cable stress for a given stress amplitude is generally calculated on the basis of 2-million cycles of fatigue test.

Table 3.1 Principle Types of Cables

Type Of Cable				
Structure	Bars ϕ 16mm	Wires ϕ 6,7 mm	Strands ϕ 15mm of 7 twisted wires	Wires with Different Profiles ϕ 2.9-7 mm
0.2% Proof Stress $\sigma_{0.2}$	1350	1470	1570-1670	-
Ultimate Tensile Strength β_Z N/mm^2	1500	1670	1770-1870	1000-1300
Fatigue $\Delta\sigma$ N/mm^2	- -	350 0.45	300~320 0.5~0.45	120~150 0.45
Modulus of Elasticit N/mm^2	210000	205000	190000 ~20000	160000 ~165000
Failure load (KN)	7624	7487	7634	7310

3.4 ANCHORAGES

In self-supporting systems, cables are anchored into structural members, such as a concrete or a steel ring, edge beam or arch, or an edge cable. In other systems, cables are to be anchored into the ground and tension anchors are required. These can be classified as

(1) Gravity anchors
(2) Rock anchors
(3) Plate anchors
(4) Tension piles

3.4.1 Gravity Anchor

This is the type of anchor most commonly selected and can be considered applicable universally. However, it is not the most economical. The reason is easily understood if one considers the magnitude of forces that must be resisted in cable roofs. It may be noted that for cables consisting of several strands or ropes it is common practice to secure these separately into the anchor.

3.4.2 Rock Anchor

In locations where firm rock is available, it is usually feasible to use the rock anchor. A dependable assessment of the strength of rock in mass is naturally a prerequisite, and would generally require a site study. If there are no serious accessibility problems, rock anchors should prove to be more economical that gravity anchors. Rock anchors can be classified as mechanical, bonded, and rock sockets.

3.4.3 Plate Anchor

Ultimate pullout loads for plate, mushroom, or other footings can be estimated approximately by existing theories. The theory considered most dependable is by Meyerof; this is based on a semi theoretical approach involving a large number of model and field tests. Meyerof has extended this work to inclined anchors under axial pull, but it has been shown that for inclinations of less that about 45 degree the pull out capacity is not affected much by change in inclination. The effect is particularly small for deep anchors, the type most commonly expected to be used for anchoring cables.

3.4.4 Tension Piles

Use of tension piles for anchoring cables is versatile alternative. Piles may be underreamed for greater for greater resistance to pull. Like a plate anchor, a pile is provided at an inclination such that the cable pulls at it axially.

3.5 FUNDAMENTAL CRITERIA FOR STRUCTURAL DESIGN

There are some fundamental items, which cannot be determined a priori as in design of normal types of structures. In the following a few considerations given to such fundamental items in the structural design is described.

3.5.1 Rigidities of Cables and Girders

One of the most important things in design of a cable-stayed roof is to find the most rational relationship between the extensional rigidity of cables and the bending rigidity of girders. Cable-stayed bridges have often experienced this kind of problem, but it is rather new in the field of building structures.

When the uniform load is acting due to dead load such as own weight, a rational state can also be found by means of erection procedures. For instance, putting temporary hinges in them during construction can control distribution of the bending moment in the girder, so that whole system is statically determinate with minimum bending moments in the girder. Another way of producing a favorable bending distribution in the girder may be to introduce tensions in the cables by means of hydraulic jacks in such a way that the bending moment in the girder is kept minimum.

3.5.2 Height of Column Anchorages

The height of column of anchorage of stay cables to the column determines slopes of the cables, and so they are important from both structural and esthetic point of view. In the structural model used in the above the bending moment in the girder and the tension in the cables are calculated against the ratio of the anchorage height to the span (h/l). When both bending moment in the girder and tensions in the cables decrease with increase in the anchorage height. They decrease very rapidly while h/l takes small values, but the trend becomes slow beyond h/l = 0.2~0.3. When h/l is very small, increase in the axial forces of the girder should also be counted.

On the other hand, to increase the anchorage height means to increase the length of both cables and column, which should also be taken in to account.

3.5.3 Lateral Stability of the Column

In the design the backstay cables are all in a vertical plane, which includes the two columns to which they are connected. This is a very unusual design, because in most examples backstays are V (or inversed V) shaped for stability of columns in the transverse direction. Stability of the columns in the transverse direction against lateral forces such as wind was very critical to the structure.

The hanging cables of the column design spread from the column tops to points of various locations on the roof, which suggests the possibility of stabilizing the columns in transverse direction. However, the stabilizing effect of the roof unit is small because of very small bending rigidity of the girders. If, for instance, a pair of horizontal forces are given at the column tops of a roof unit in the transverse direction 60% of these forces have to be treated by the bending of the columns. This is not favorable to the columns, since they are provided with amount of rotational resistance in the transverse direction as well.

This situation is greatly improved when the adjacent roof units are connected to each other at their mutual boundaries in the vertical direction (they free to make relative movements in the horizontal direction there for free expansion). The deformation of the roof units subjected to the column-top forces is then restrained by the other units around it, and the portion of horizontal forces to be treated by the columns thus reduce to only 20%.

CHAPTER 4 NON-LINEAR BEHAVIOUR OF CABLE-SUPPORTED ROOF

4.1 GENERAL

A large span cable-supported roof is a relatively flexible and an extended in plane structural system in which the girder is elastically supported at joints along its length by inclined cable-stays. Large displacements occur in such a flexible structure under normal design loads such as dead and live loads. The conventional linear structural analysis methods can not handle such a geometrically non-linear problem having large displacements and small stains. The most popular approach in performing the non-linear static analysis is the stiffness method. This approach is widely used by analysts and researchers since it can be conveniently adopted for computer use. It can also be easily applied to almost any type of structural system with various types of individual structural elements.

A fundamental difference between geometrically linear and geometrically non-linear analysis is that in linear analysis equilibrium is satisfied on the initial undeformed configuration, whereas in nonlinear analysis equilibrium must be satisfied in the deformed configuration. To achieve final equilibrium in a non-linear analysis, solving the problem many times, constantly adjusting the applied forces based on the current state of equilibrium, and modifying the geometry based on the current displacements, until reduce the residual forces to an acceptable level.

4.2 TYPES OF NONLINEARITY

In structural mechanics, material may yield or creep; local buckling may arise; gaps may open or close. Nonlinear problems pose the difficulty of describing phenomena by realistic mathematical and numerical models and the difficulty of solving nonlinear equations that result. Effort required of the analyst increases substantially when a problem becomes nonlinear.

In structural mechanics, types of nonlinearity include the following.

(1) **Material nonlinearity**, in which material properties are functions of the state of stress or strain. Examples include nonlinear elasticity, plasticity, and creep.

(2) **Contact Nonlinearity**, in which a gap between adjacent parts may open or close, the contact are between parts changes as the contact force changes, or there is sliding contact with frictional forces.

(3) **Geometric Nonlinearity**, in which deformation is large enough that equilibrium equation, must be written with respect to the deformed structural geometry. Also, loads may change direction as they increase, as and when pressure inflates the membrane.

4.3 NONLINEAR BEHAVIOUR OF CABLE SUPPORTED ROOF

Very few researches have performed the three-dimensional analysis. Fleming developed an algorithm for 3-D analysis using cable nonlinearity only. For including beam-column nonlinearity for girder and masts, the stability functions are extended in the present investigation. Moreover, the tangent stiffness matrix of cable as well as mast and girder elements is computed by using large deflection theory and non-linear strain-displacement relationships. For speeding up the convergence rate the mixed procedure out lined in step iterative method is used. In this chapter, the detail formulation along with the brief description of various methods is presented for completeness.

For a linear structural system, the static displacements can be easily computed by solving the set of simultaneous stiffness equations, expressed in matrix form,

$$[K]\{D\} = \{P\} \qquad \text{........ (4.1)}$$

In which, $[K]$ is the global stiffness matrix of the structure, $\{D\}$ is the vector of joint displacements, and $\{P\}$ is the vector of applied joint loads.

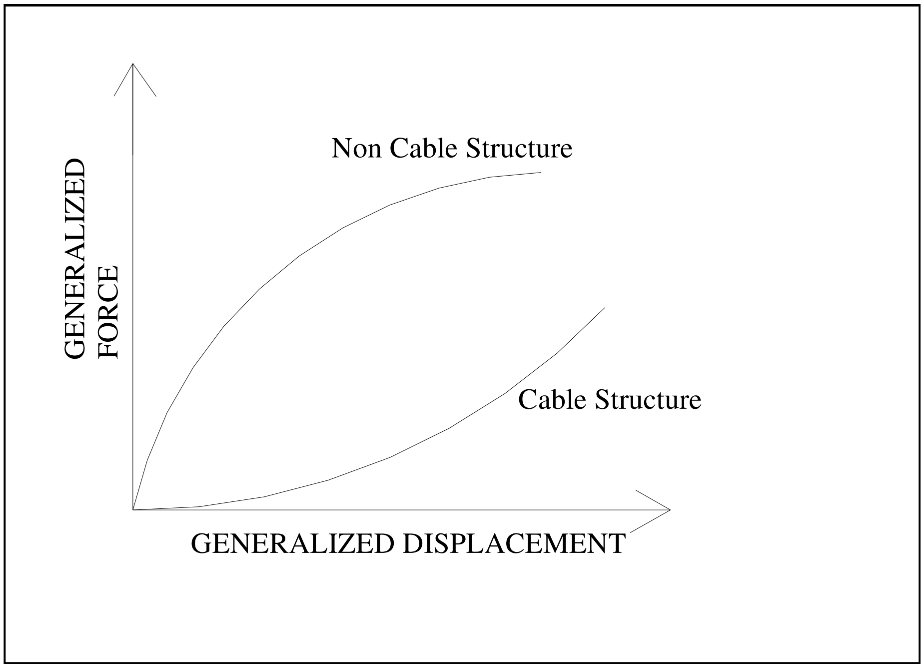


Fig. 4.1 Non-linear Force Displacement Relationship

For a non-linear structural system, the stiffness of the structure changes as the structures deforms; therefore, the terms in the matrix [K] change as the load is applied. The non-linear force-displacement relationship for cable and non-cable structures is as shown in (fig. 4.1) In non-cable structures also called softening structural systems, the stiffness of the structure decreases with increasing deformation. In such a case, the axial compression in the structural elements, under applied loads, reduces their bending stiffness. The cable structures are hardening structural systems in which the stiffness of the structure increases with increasing joint displacements.

For the nonlinear stiffness matrix of a truss element or a beam element consists of two components:

$$K = K_0 + K_1 \qquad \text{……. (4.2)}$$

Where,

K_0 = Elastic Stiffness Matrix

K_1 = Geometric Stiffness Matrix

The elastic and geometric stiffness matrices are determined for each element at the beginning of each iteration and assembled to form the system stiffness matrix.

4.4 NONLINEAR SOLUTION PROCEDURES

There are three basic techniques for solving the nonlinear equations (4.1), when the stiffness $[K]$ varies as a function of nodal displacements and member forces.

4.4.1 Incremental or Step by Step Method

In this method, the external load is applied in several small steps and the structure is assumed to respond linearly within each step. At the beginning of each new step, the stiffness of the structure is recomputed based on the structural geometry and member end actions at the end of the previous load step. This new stiffness is used to compute the displacement increments corresponding to the new load increments. The final displacements and member forces, after applying all load increments, are obtained by adding the incremental displacements and member forces that correspond to all load increments.

This procedure is simple to apply and has been widely used, since no iterations are required during the whole analysis. The disadvantage of this method is that errors are likely to accumulate after several steps unless very fine steps are used. The solution may, therefore, diverge considerably from response as shown in fig. 4.2.

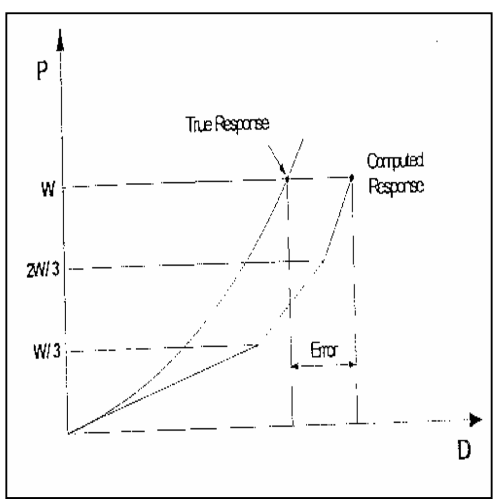


Fig. 4.2 Incremental Procedure

$$K_{i-1}\, d\ \Delta_1 = dP_i \qquad \ldots\ldots\ (4.3)$$

K_{i-1} is the tangent stiffness matrix. The total deflection is

$$\Delta i = \Delta_0 + \sum_{j-1}^{n} d\Delta j \qquad \ldots\ldots\ (4.4)$$

Where Δ_0 is the initial deflection and n = no. load steps upto $(i-1)$ load step.

4.4.2 Iterative or Newton-Raphson Methods

Two types of iterative methods are commonly used, namely Newton-Raphson and Modified Newton-Raphson methods.

In Newton-Raphson approach (fig.4.3) the total load is in one increment. The displacements are initially computed using the tangent stiffness of the undeformed structure. The stiffness is then recomputed corresponding to this deformed shape before the member end loads is computed. Since the final stiffness, which is used to compute the member end loads, differs from the initial stiffness used to compute the joint displacements, equilibrium will not be satisfied and unbalanced loads will exist at the joints. These unbalanced loads are next applied as a new set of joint loads, with the corresponding change in displacements being computed using the stiffness corresponding to the new deformed position of the structure. The final solution can be obtained by iterating until the unbalanced loads at the end of a cycle are smaller than an acceptable tolerance limit.

The disadvantage of Newton-Raphson method is that a large amount of computational effort may be required form and decompose the stiffness matrix at each iteration cycle. In Modified Newton-Raphson (fig. 4.3) approach, the stiffness matrix is formed and decomposed only once. However, this modified method generally converges more slowly than Newton-Raphson method.

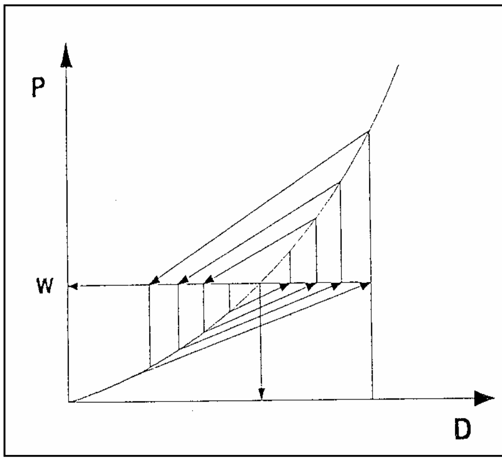


Fig. 4.3 Modified Newton-Raphson Method (Converging Case)

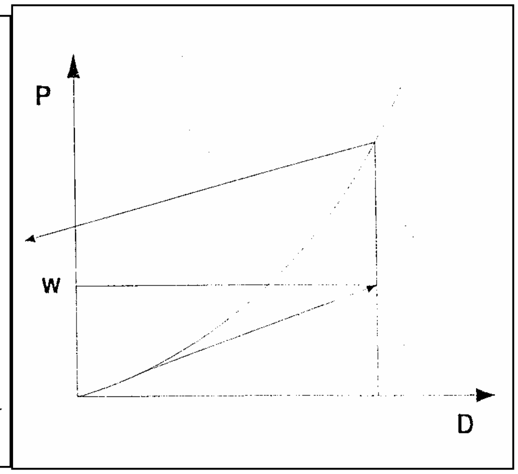


Fig. 4.4 Modified Newton Raphson Method (Diverging Case)

Modified Newton-Raphson may never converge in some cases (fig.4.3), particularly when the nonlinearity is strong, and the load increment is relatively large. Therefore, it is not recommended to use Modified Newton-Raphson method in the nonlinear static analysis of cable supported roofs. Nevertheless, it can be used, within each time step, in the nonlinear dynamic analysis of cable supported roofs when the time step is small (Δt ~0.01 to 0.02 seconds) and the expected variations in displacements during the time step is not large.

For geometric non-linear problems the Newton-Raphson iterative technique is very effective. The basic steps with respect to Fig. 4.5 are as follows:

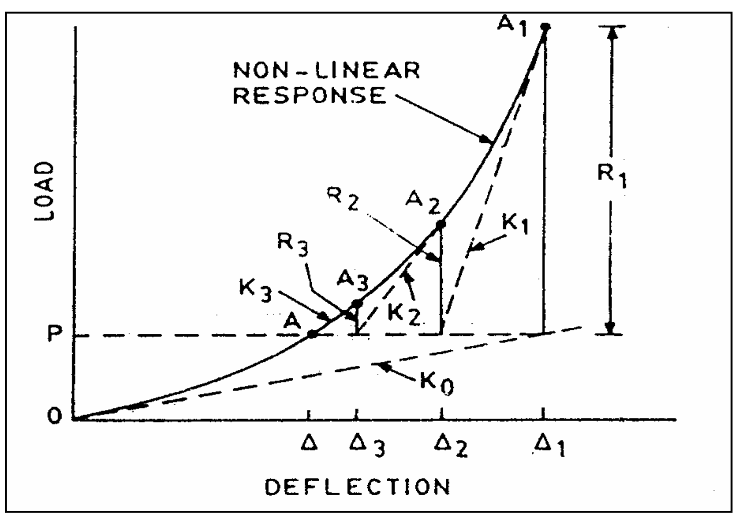


Fig. 4.5 Iterative Method

Step 1 Compute the value of Δ_1 from $K\Delta_1 = P$.

Step 2 Compute the unbalanced load vector R_1 using the initial stiffness.

Step 3 Compute the tangent stiffness K_1 at A_1 and determine Δ_2 using the equilibrium equation

$$K\Delta = P - R \quad \text{........ (4.5)}$$

Step 4 Compute the unbalanced load vector R_2 on basis of values obtained in step 3.

Step 5 Repeat step 3 and 4 until the convergence is achieved, that is, the values of R or Δ in two successive cycles agree within the desired accuracy.

4.4.3 Step-Iterative or Mixed Method

In the Mixed method – I, the load is applied incrementally, but after each increment, successive iterations are performed using either Newton-Raphson or Modified Newton-Raphson iteration within each load step as shown in fig. 4.6.

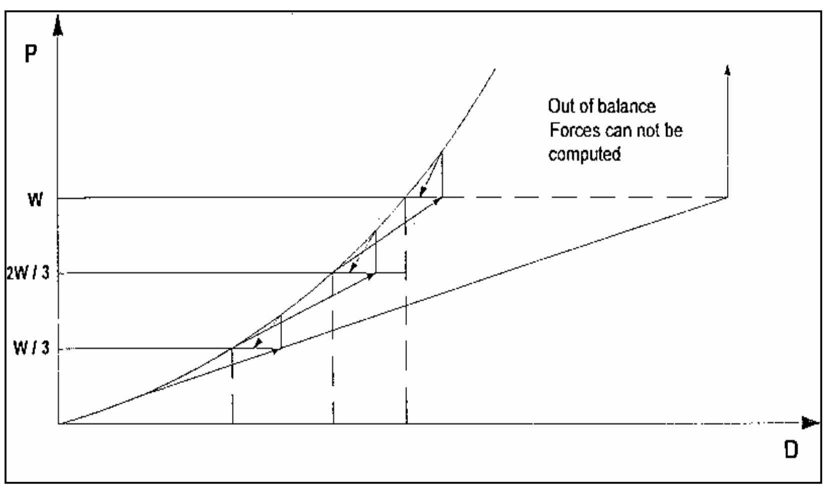


Fig. 4.6 Mixed Procedure I

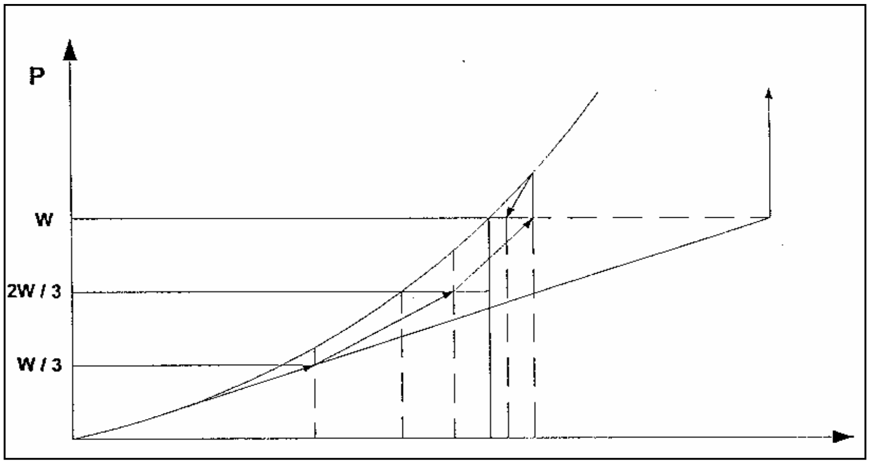


Fig. 4.7 Mixed Procedure II

In the Mixed method – II, the unbalanced joint loads are applied incrementally during each cycle of iteration. The stiffness matrix of the structure is recomputed at the beginning of each load increment. At the end of each iteration cycle, the unbalanced loads at joints are recomputed and applied incrementally during the next cycle. This procedure is shown in fig. 4.7, where three load steps are used during the first cycle and only one load step is used for the successive iteration cycle.

The basic steps with respect to fig. 4.8 are as follows:

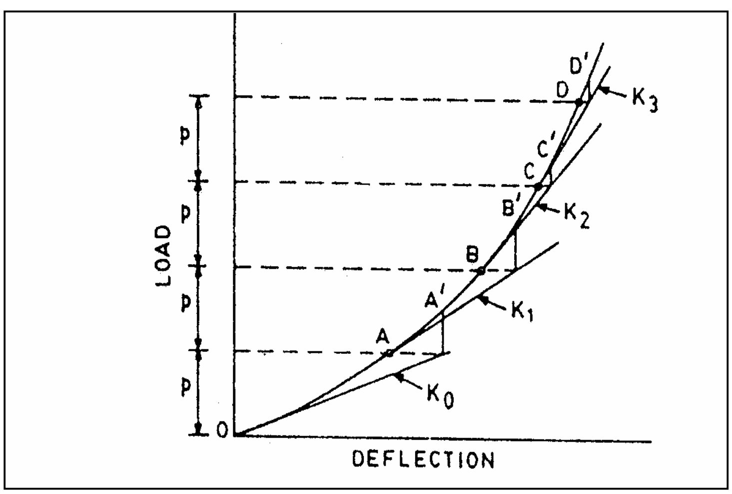


Fig. 4.8 Incremental cum Iterative Method

Step 1 Apply a small load p, assume initial stiffness K_0 and locate point $A^{'}$ on the response curve. This gives the unbalanced load R.

Step 2 Iterate is using the tangent stiffness matrix at $A^{'}$ in the first iteration till point A is reached.

Step 3 Apply the next load increment, assume tangent stiffness K_1 at point A, and locate point $B^{'}$ on the response curve.

Step 4 Repeat step 2 and 3, till all the load increments are exhausted.

It is important to note that the load in each incremental step need not be equal.

4.5 SOURCES OF NONLINEARITY OF CABLE SUPPORTED ROOF

In spite of the fact the behavior of the material of the structural elements in a cable supported roof is linear-elastic, the overall load displacement relationship for the structure is nonlinear under normal design loads. The primary sources of nonlinearity are three viz.:

(1) Sag in the inclined cables due to their own self weight.

(2) Combined axial force and bending moment in girder and mast element.

(3) The geometry change caused by the large displacements.

4.5.1 Nonlinear Behaviour of Cables

When a cable is supported from its ends, and subjected to its own weight and an externally applied axial tensile force, it sags into the shape of catenaries. The axial stiffness of the cable varies nonlinearly as a function of end displacements, since part of the end movement occurs due to material deformation and another part occurs due to change in sag. As the axial tension increases, the cable sag becomes smaller and smaller, and the end movement occurs mainly due to material deformation. This explains why the apparent axial stiffness of the cable increases as its tensile stress increases (fig.4.1).

4.5.2 Nonlinear Behaviour of Girder and Mast

When assuming deformations in any structural system, the axial and flexural stiffness of the bending members are usually do not coupled. However, an interaction does exist between the axial and flexure deformations, since the transverse deflections of a member act as a moment arm for the simultaneously applied axial load. The moment resulting from this flexural deformations-axial force interaction either augments or reduces the original flexural bending moment in the member. The effect of this interaction is that the effective flexural stiffness of the member decreases for a compressive axial force and increases for a tensile force. In addition, the axial stiffness is affected by the bending deformation due to the change in the chord length as the member bends.

4.5.3 Geometry Change due to Large Displacements

In linear structural analysis, it is assumed that the joint displacements of the structure under the applied loads are negligible with respect to the original coordinates. Thus, the geometric changes in the structure can be ignored and the overall stiffness of the structure in the deformed shape can assumed to be equal to the stiffness of the unreformed structure. However, in cable-supported roofs, large displacements can occur under normal design loads, which ultimately lead to significant change in the roof geometry. In such a case, the stiffness of the roof in the deformed shape should be computed from the new geometry of the structure.

CHAPTER 5 MODELING OF CABLE-SUPPORTED ROOF IN STAAD.PRO 2003

5.1 GENERAL

Now a days many softwares are programmed to include the stiffness analysis method. The use of STAAD.Pro 2003 is done for the design problem chosen here. STAAD.Pro -2003 can analyze problems of many fields mainly of structures. The problems can be analyzed for many conditions like static, transient, modal analysis, response spectrum analysis, P-delta analysis, nonlinear analysis, cable analysis etc.

5.2 STARTING WITH STAAD.Pro

Stepwise procedure for modeling of cable-supported roof and analysis of cable static linear as well as static nonlinear are as follow.

- Starting the Program
- Creating New Structure
- Creating Joints and Members
- Specifying Member Properties
- Specifying Material Constants
- Specifying Supports
- Specifying Initial Tension and Member Truss Command
- Specifying Loads
- Specifying the Analysis Type
- Performing Analysis
- Viewing the Output Files

5.2.1 Starting the Program

Select the STAAD.Pro icon from the STAAD.Pro 2003 program group.

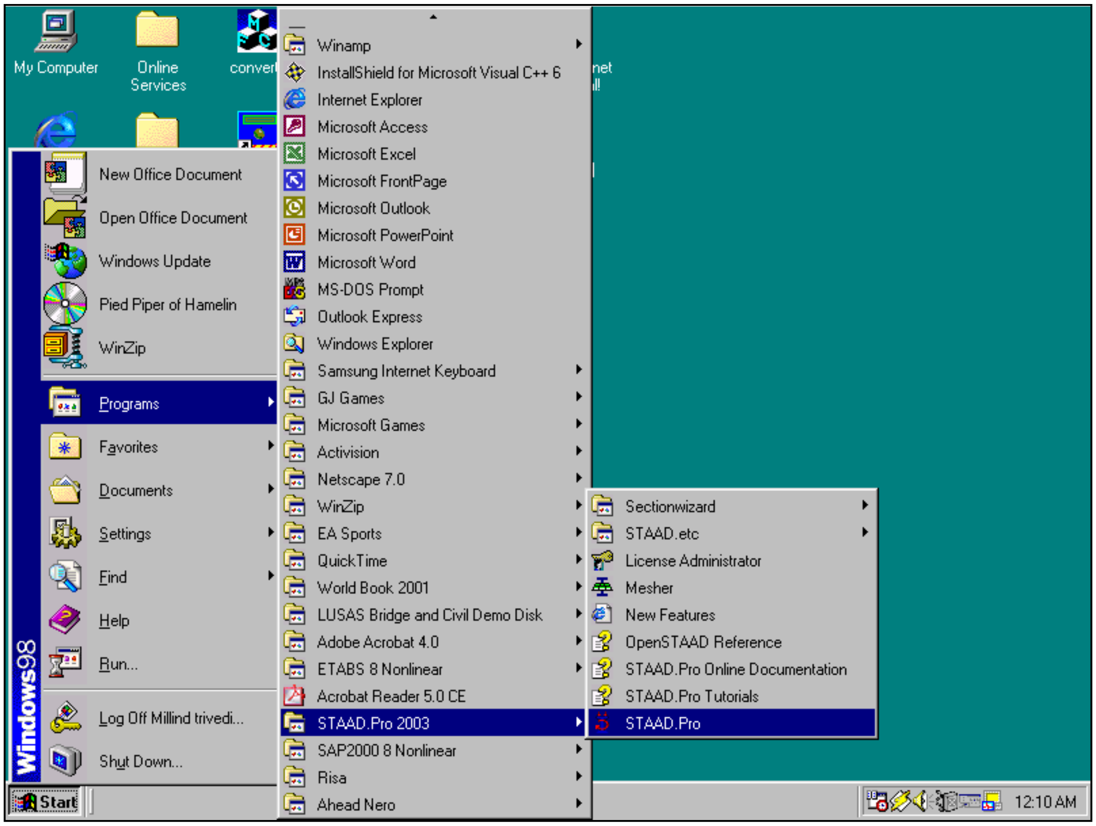


Fig. 5.1

The STAAD.Pro Graphical Environment will be invoked and the following screen comes up as in fig. 5.2.

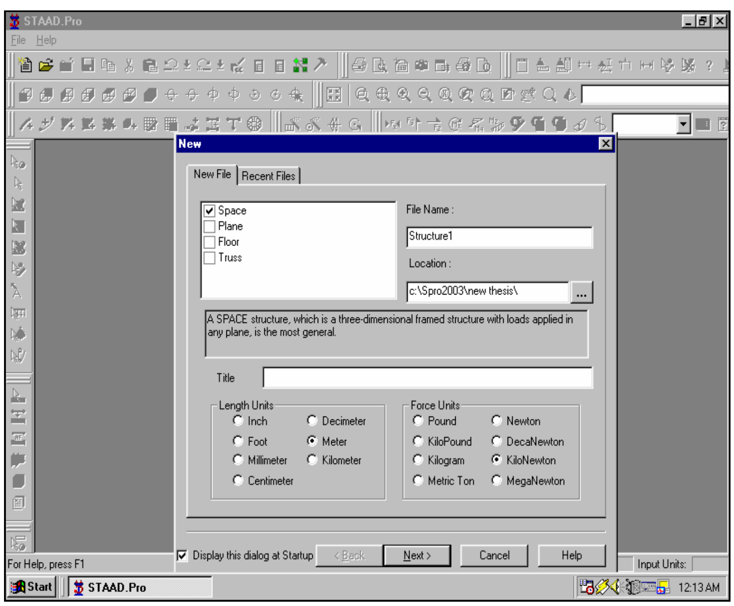


Fig. 5.2

Note about the unit system:

There are two base unit systems in the program which control the units (length, force, temperature, etc.) in which, values, specifically results and other information presented in the tables and reports, are displayed in. The base unit system also dictates what type of default values the program will use when attributes such as Modulus of Elasticity, Density, etc., are assigned based on material types – Steel, Concrete, Aluminum – selected from the program's library. These two unit systems are English (Foot, Pound, etc.) and Metric (kN, Meter, etc.) Unit system can be change from File / Configure Menu as shown in Fig. 5.3.

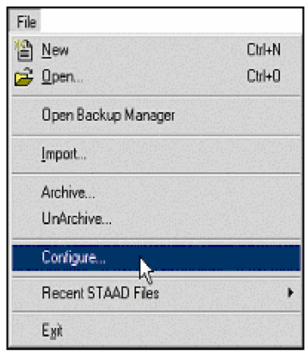


Fig. 5.3

For the modeling of cable-supported roof structure Metric System is used as shown in Fig. 5.4.

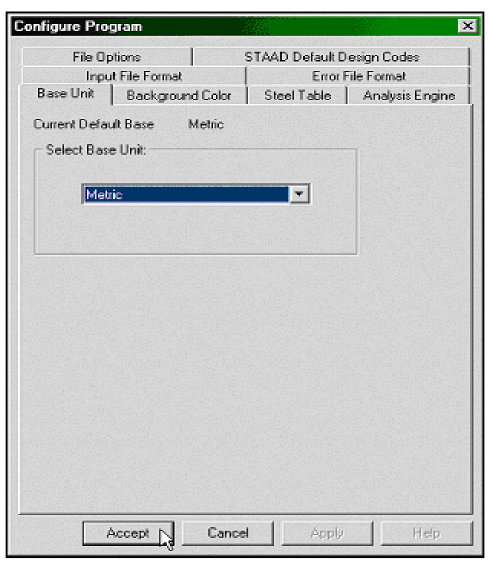


Fig. 5.4

Click on the Accept button to close the above dialog box. Following this, select File | New (Fig. 5.5)

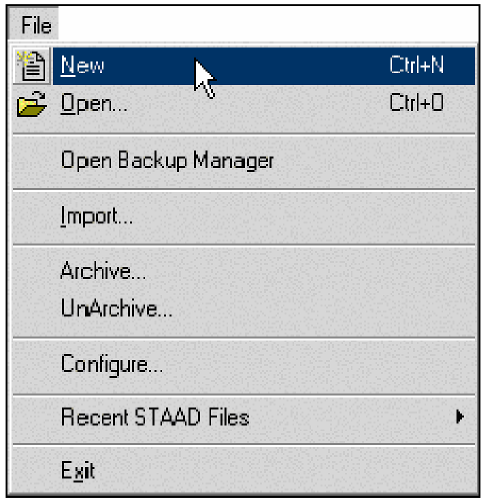
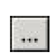


Fig. 5.5

5.2.2 Creating a New Structure

In the New dialog box, we provide some crucial initial data necessary for building the model. The structure type is to be defined by choosing from among Space, Plane, Floor and Truss. A Space type is one where the structure, the loading or both, cause the structure to deform in all 3 global axes (X, Y and Z). In a Plane type, the geometry, loading and deformation are restricted to the global X-Y plane only. A Floor type is a structure whose geometry is confined to the X-Z plane. A Truss type of structure carries loading by pure axial action. Truss members are deemed incapable of carrying shear, bending and torsion. For our model, let us choose Space.

Provide a name in the File Name edit box. (Fig. 5.6) This is the name under which the structure data will be saved on the computer hard disk. The name “Structure?” (? Will be a number) is recommended by the program by default, but we can change it to any name we want. Let us choose the name Cable.

The program under Location provides a default path name -the location on the computer drive where the file will be saved. If one wishes to save the file in a different location, type in the name, or click the  button and specify the desired path.

An optional title for the project may be entered in the Title edit box. Let us give it the title Cable Structure.

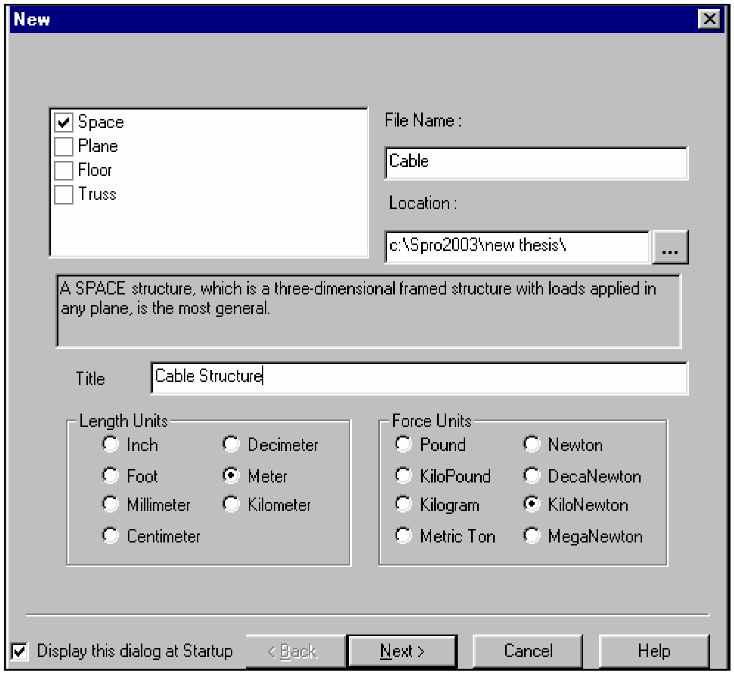


Fig. 5.6

In the next dialog box, we choose the tools to be used initially to construct the model. Add Beams, Add Plates or Add Solids are, respectively, the starting points for constructing beams, plates or solids.

For cable-supported roof model, let us check the Add Beam option (Fig 5.7). Click on the Finish button. The dialog box will be dismissed and the STAAD.Pro graphical environment will be displayed.

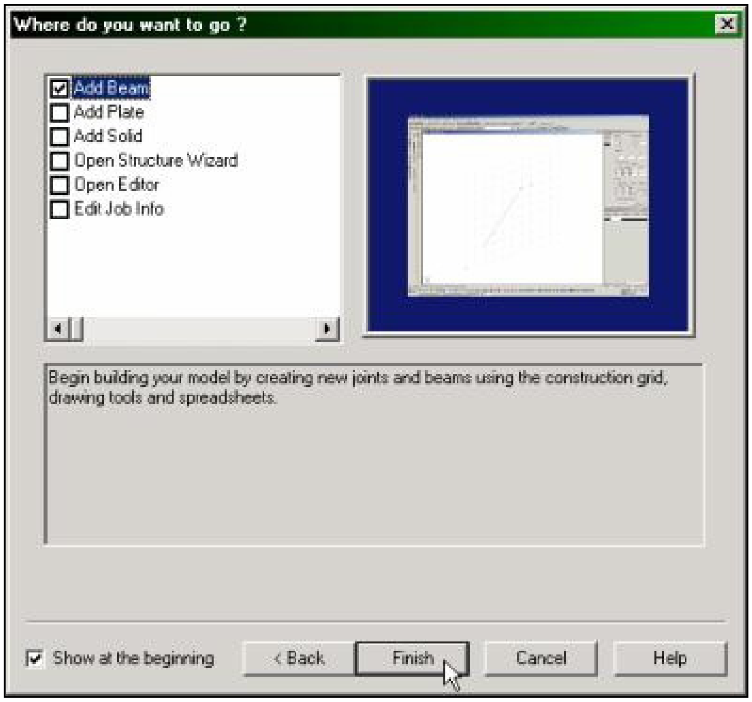


Fig. 5.7

5.2.3 Building the STAAD.Pro Model

Now be ready to start building the model geometry. The steps and, wherever possible, the corresponding STAAD.Pro commands (the instructions which get written in the STAAD input file) are described in the following sections.

5.2.3.1 Generating the model geometry

The structure geometry consists of joint numbers, their coordinates, member numbers, the member connectivity information, plate element numbers, etc. From the standpoint of the STAAD command file, the commands to be generated are:

JOINT COORDINATES

1 135.868 167.713 0; 2 144.435 168.57 0; 3 153.003 169.426 0;

MEMBER INCIDENCES

1 1 509; 2 2 31; 3 3 34; 4 4 37; 5 6 521; 6 8 39; 7 7 42; 8 9 38; 17 1 18;

One selected the Add Beam option earlier to facilitate adding beams to create the structure. This initiates a grid in the main drawing area as shown in Fig. 5.8. The directions of the global axes (X, Y, Z) are represented in the icon in the lower right hand corner of the drawing area.

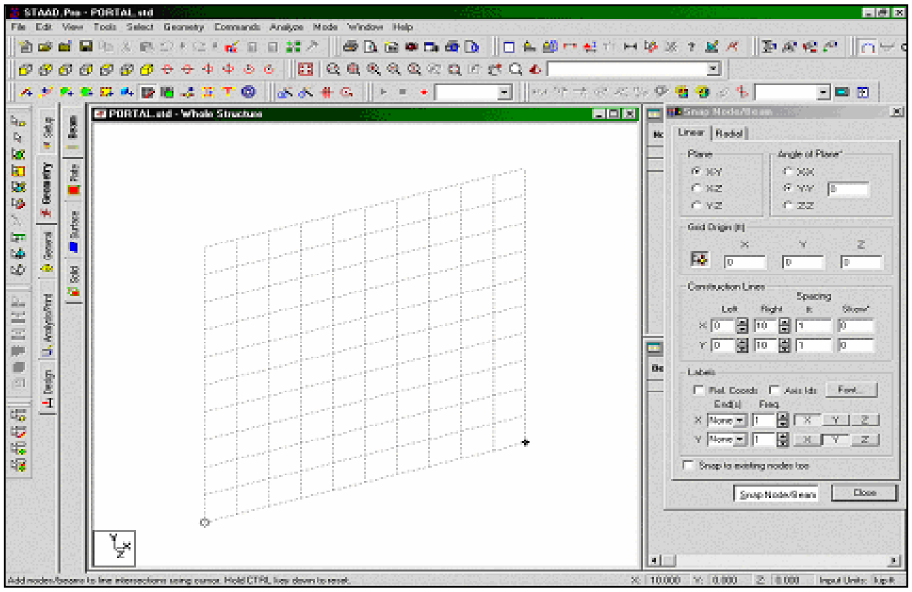


Fig. 5.8

A Snap Node/Beam dialog box also appears in the data area on the right side of the screen. The size of the model that can be drawn at any time is controlled by the number of Construction Lines to the left and right of the origin of axes, and the Spacing between adjacent construction lines. By setting 60 as the number of lines to the right of the origin along X, 91 above the origin along Y, and a spacing of 1meter between lines along both X and Y (see Fig. 5.9) we can draw a frame 60m X 91m, adequate for our structure. Please note that these settings are only a starting grid setting, to enable us to start drawing the structure, and they do not restrict our overall model to those limits.

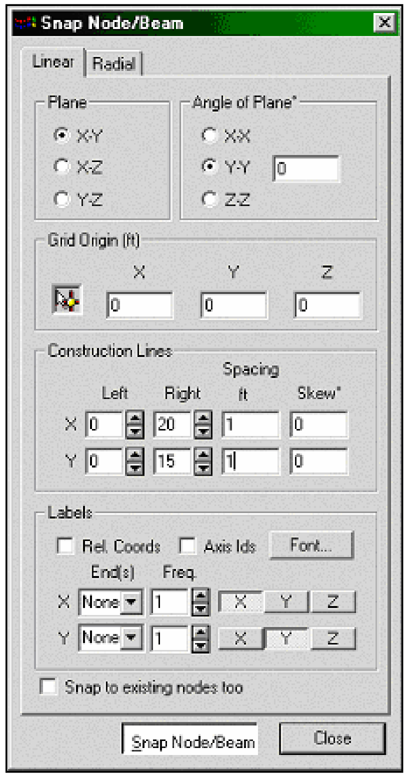


Fig. 5.9

With the help of the mouse, click at the origin (0, 0) to create the first node as shown in Fig. 5.10.

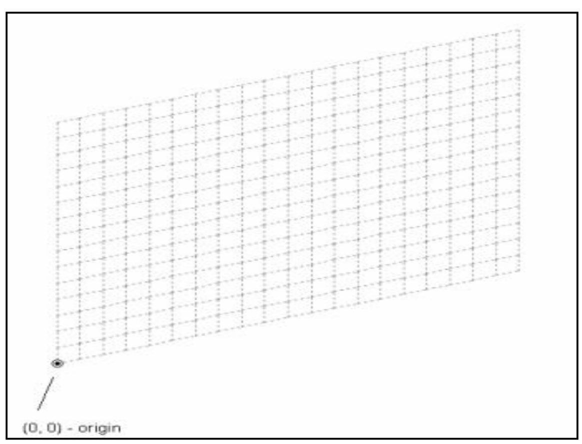


Fig. 5.10

In a similar fashion, click on the remaining points to create nodes and automatically join successive nodes by beam members.

After Completed to join all nodes and members, the structure will be displayed in the drawing area as shown in Fig. 5.11.

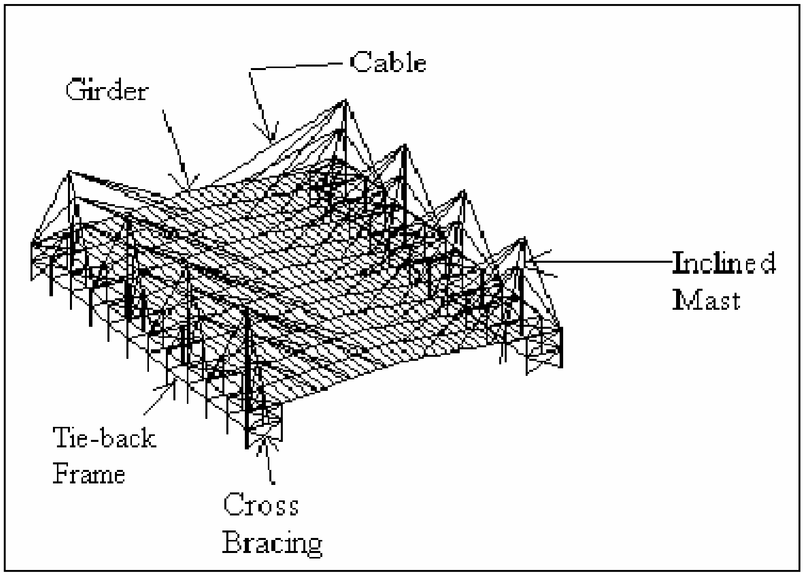


Fig. 5.11

5.2.3.2 Specifying Element Properties

Next task is to assign cross section properties for the beams and columns. For those of one should curious to know the equivalent commands in the STAAD command file, they are:

MEMBER PROPERTY INDIAN

1 TO 8 17 19 29 TO 48 TABLE ST ISMB600

988 TO 1107 PRIS YD 0.025

To define member properties, click on the Property Page icon located on the top toolbar.

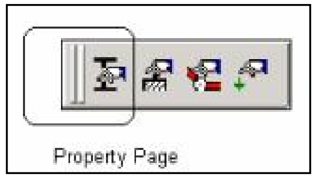


Fig. 5.12

In either case, the Properties dialog box comes up (see Figure 5.14). Since If know that the first property (ST ISMB 600) is to be assigned to members 1 to 8, let us first select these members prior to generating the property itself. Select Members 1 to 8 (horizontal members) by holding the ‘Ctrl’ key down and clicking on them. The selected members will be highlighted.

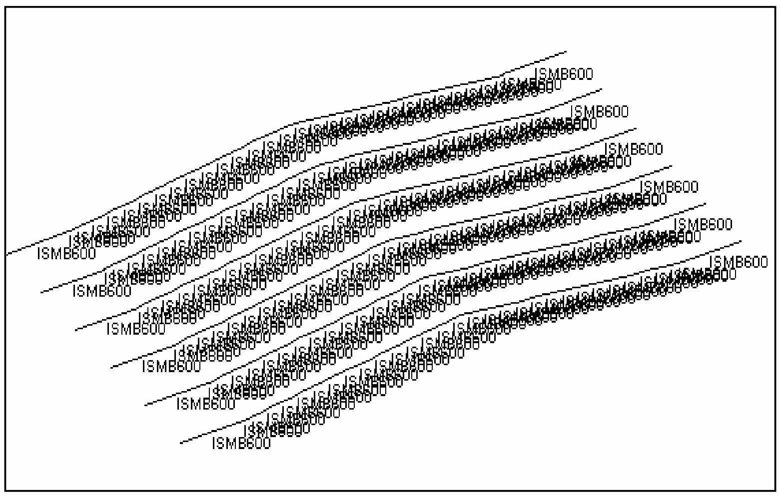


Fig. 5.13

The property type we wish to create is the ISMB shape from the INDIAN table. This is available under the Database button in the Properties dialog box as shown below in Fig. 5.14.

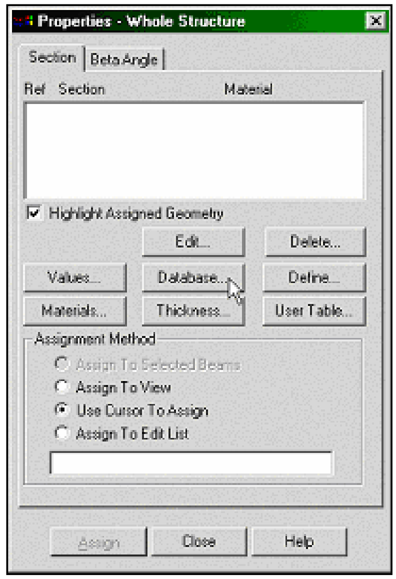


Fig. 5.14

In the Select Country dialog box that appears as shown in Fig. 5.15, choose the country name whose steel table you want to use, in our case, Indian. Then, click on OK.

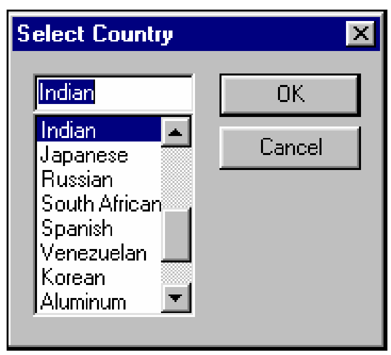


Fig. 5.15

In the Indian Steel Table dialog box as shown in Fig. 5.16, select the S Shape tab. Notice that the Material box is checked. Let us keep it that way because it will enable to subsequently assign the material constants E i.e. modulus of elasticity, Density, Poisson Ratio, etc. along with the cross-section since one has want to assign the default values.

Choose ISMB 600 as the beam size, ST as the section type. Since one has already selected the members that are to be associated with this property, let us click on the Assign button as shown in the fig. 5.16.

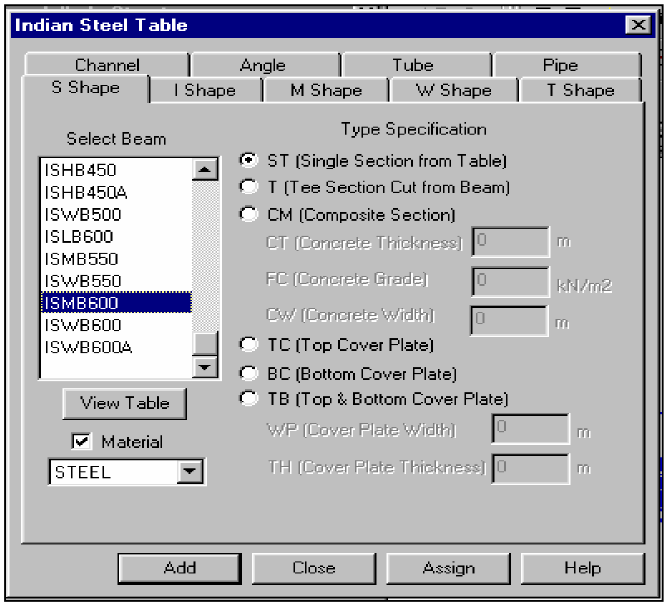


Fig. 5.16

After the first member property has been assigned, the model will look as shown in Fig. 5.17

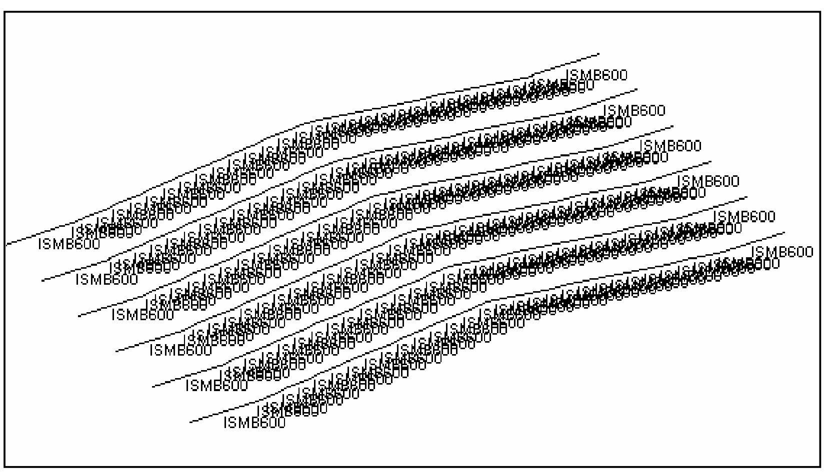


Fig. 5.17

For Cable members one should assign properties in the STAAD command file are:

MEMBER PROPERTY

892 TO 1011 PRIS YD 0.025

In the dialog box that comes up, select the Circle tab. Notice that the field called Material is presently on the checked mode. If one keeps it that way, the material properties of cable (E modulus of elasticity, Poisson ratio, Density, Alpha, etc.) will be assigned along with the cross-section name. The material property values so assigned will be the program defaults. Then, enter the following values:

YD = 0.025 m

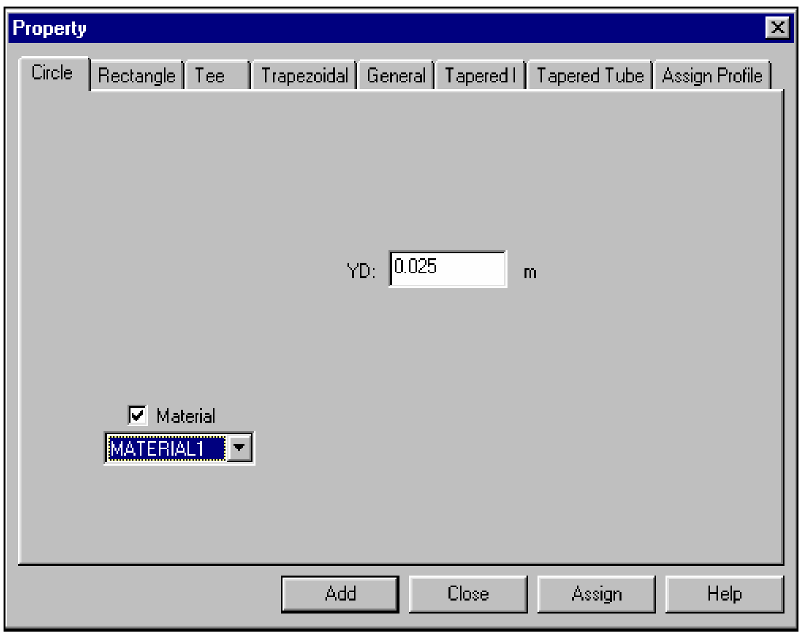


Fig. 5.18

To associate the second member property, repeat the same procedure until all members are assigned.

5.2.3.3 Specifying Member Properties

The material constants got assigned to the members along with the properties, and the following commands were generated in the command file:

CONSTANTS

E 2.05e+008 MEMB 1 TO 891

POISSON 0.3 MEMB 1 TO 891

DENSITY 76.8195 MEMB 1 TO 891

ALPHA 6.5e-006 MEMB 1 TO 891

Hence, there is no longer a need to assign the constants separately. However, one could go to the menu option Commands | Material Constants and assign them explicitely as shown in the Fig.5.19.

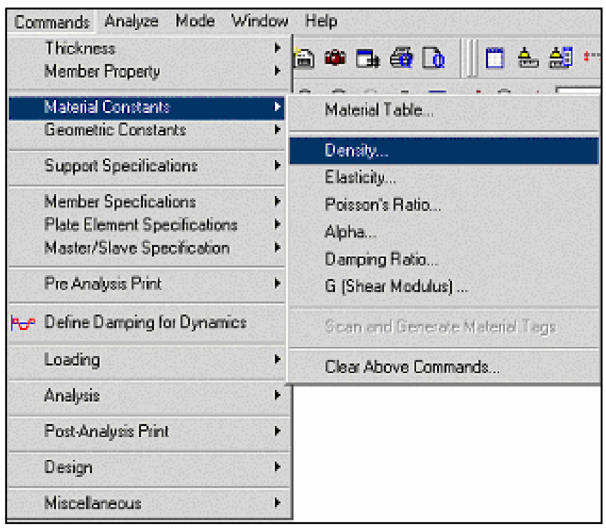


Fig. 5.19

Especially for cable member one should have option to select defined material. In which user have freedom to choose material properties like modulus of elasticity, poisson’s ratio, density, etc, as shown in fig. 5.20

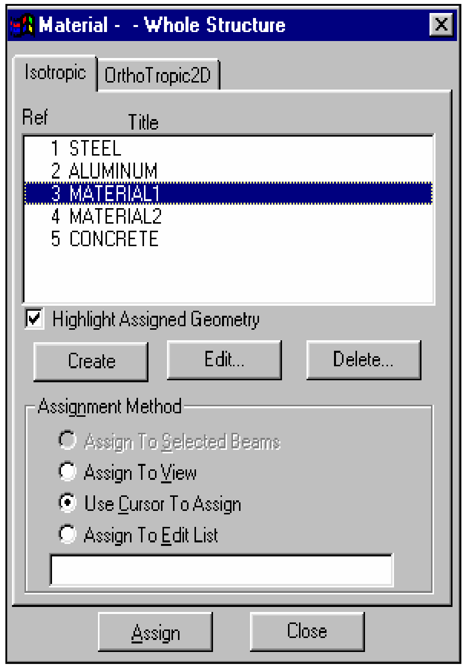


Fig. 5.20

MATERIAL 1 is defined by click on the create button which shows the dialog box as shown in Fig. 5.21

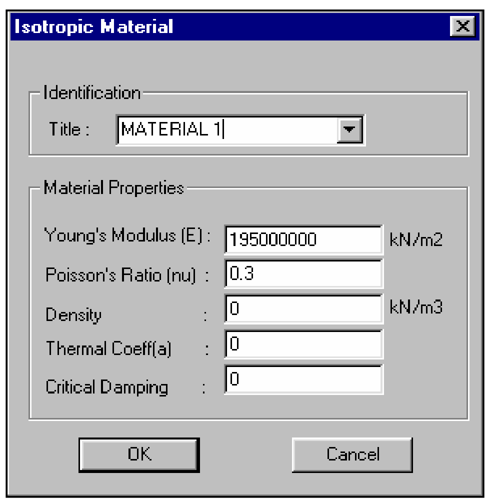


Fig. 5.21

5.2.4 Specifying Support

The base nodes of all the columns are restrained against translation and rotation about all the 3 global axes. In other words, fixed supports are to be specified at those nodes. The commands to be generated are:

SUPPORTS

11 13 TO 15 50 52 TO 54 FIXED

To create supports, click on the Support Page icon located in the Structure Tools toolbar as shown below in fig. 5.22.

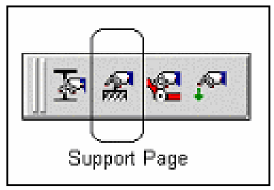


Fig. 5.22

Alternatively, one may go to the General | Support Page from the left side of the screen.

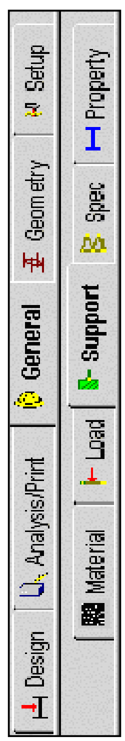


Fig. 5.23

In either case, the Supports dialog box comes up as shown in Figure 5.24. Since already know that nodes 11, 13 to 15 are to be associated with the fixed support, using the Nodes Cursor , select these nodes.

Then, click on the Create button in the Supports dialog box as shown in fig. 5.24.

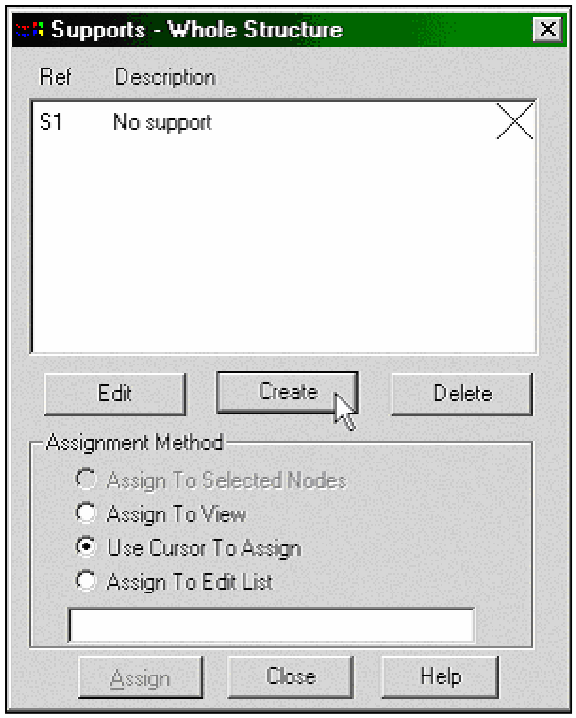


Fig. 5.24

The Create Support dialog box comes up. In the dialog box, the Fixed tab happens to be the default which is convenient for this case. Click on the Assign button as shown in fig.5.25

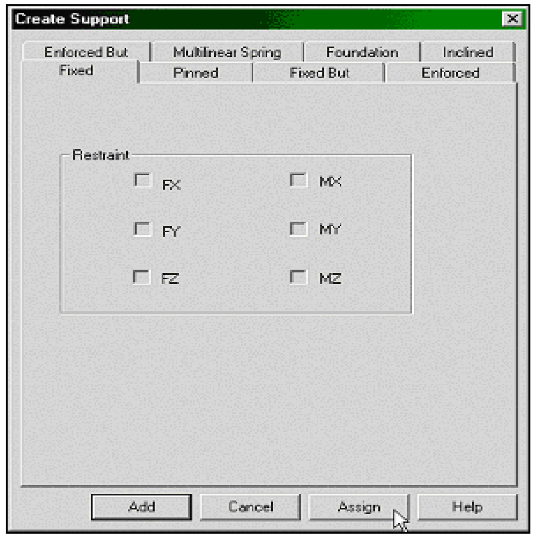


Fig. 5.25

After the supports have been assigned, the structure will look like the one shown below.

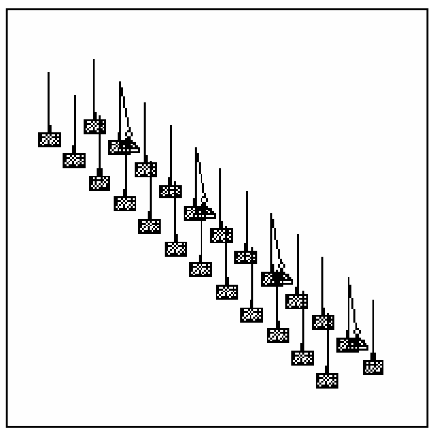


Fig. 5.26

5.2.5 Specifying Member Truss and Initial Tension

5.2.5.1 Purpose

The Member Specifications menu option in the Commands menu allows the user to define various structural conditions of the members such as cable, truss, tension-only, compression-only, member release, member offset, etc.

5.2.5.2 Description

The Member Specifications menu option offers several sub-menu options as shown below. These options are described in the following pages.

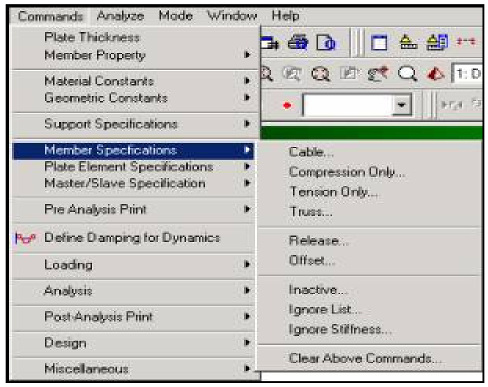


Fig. 5.27

When you select the Cable menu option, the Beam Specs dialog box appears, as shown below.

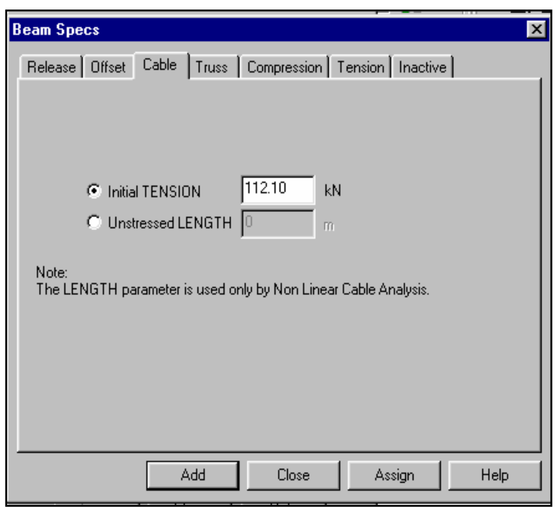


Fig. 5.28

Provide the initial tension in the cable. Provide either the Initial TENSION in the cable as a force, or the Unstressed LENGTH of the cable member.

The commands to be generated are:

MEMBER TRUSS

892 TO 1011

MEMBER CABLE

892 TO 1011 TENSION 112.10

5.2.5.3 Linearized Cable Members

Using the MEMBER CABLE command may specify Cable members. While specifying cable members, the initial tension in the cable must be provided.

The increase in length of a loaded cable is a combination of two effects. The first component is the elastic stretch, and is governed by the familiar spring relationship

$$F = Kx \quad \text{where} \quad K_{elsatic} = \frac{EA}{L} \qquad \text{... (5.1)}$$

The second component of the lengthening is due to a change in geometry (as a cable is pulled taut, sag is reduced). This relationship can be described by

$$F = Kx \quad \text{but} \quad K_{sag} = \frac{12T^3}{w^2L^3}\,\frac{(1.0)}{\cos^2\alpha} \qquad \text{… (5.2)}$$

Where w = Weight per unit length of cable

T = Tension in Cable

α = angle that the axis of the cable makes with a horizontal plane (= 0, cable is horizontal; = 90, cable is vertical)

Therefore, the stiffness of a cable depends on the initial installed tension (or sag). These two effects may be combined as follows.

$$K_{comb} = \frac{1}{1/K_{sag} + 1/K_{elastic}} \qquad \text{… (5.3)}$$

$$K_{comb} = \frac{EA/L}{\left[1 + w^2 L^2 EA(\cos^2 \alpha)/12T^3\right]} \quad \ldots (5.4)$$

When T = infinity, $K_{comb} = EA/L$

When $T = 0$, $K_{comb} = 0$

It may be noticed that as the tension increases (sag decreases) the combined stiffness approaches that of the pure elastic situation.

5.2.5.4 Non-linear Cable Member

Cable members for the Non Linear Cable Analysis may be specified by using the MEMBER CABLE command. While specifying cable members, the initial tension in the cable or the unstressed length of the cable must be provided. The user should ensure that all cables will be in sufficient tension for all load cases to converge. Use selfweight in every load case and temperature if appropriate; i.e. don't enter component cases (e.g. wind only).

There are two cable types in the nonlinear PERFORM CABLE ANALYSIS procedure. One is the cable whose stiffness is described in section 5.2.5.4 above. In the nonlinear cable analysis, the cable may have large motions and the sag is checked on every load step and every equilibrium iteration. The second cable type is not available yet. It is a catenary shaped cable that is integrated along its length to determine its end forces on every load step and every equilibrium iteration.

In addition there is a nonlinear truss, which is specified in the Member Truss command. The nonlinear truss is simply any truss with pretension specified. It is essentially the same as a cable without sag. This member takes compression. If all cables are taut for all load cases, then the nonlinear truss may be used to simulate cables. The reason for using this substitution is that the truss solution is more reliable.

Notes

The TRUSS member has only one degree of freedom-the axial deformation. It is not equivalent to a frame member with moment releases at both ends. Note also that Member

Releases are not allowed. Selfweight and transverse loads may induce shear/moment distributions in the member.

5.2.6 Specifying Load Cases

Six load cases are to be created for this structure as a primary load cases. Notice that cases 7 to 31 are to be generated not as the standard combination type, but using a combination load type called REPEAT LOAD.

This requires analyzing this structure using an analysis type called Perform Cable Analysis. A Perform Cable Analysis is a non-linear type of analysis. In STAAD, to accurately account for the Cable Analysis effects arising from the simultaneous action of previously defined horizontal and vertical loads; those previous cases must be included as components of the combination case using the REPEAT LOAD type.

```
UNIT METER KN

LOAD 1 DEAD LOAD
SELFWEIGHT Y -1
JOINT LOAD
18 20 447 449 FY -3.58

LOAD 2 LIVE LOAD
JOINT LOAD
18 20 447 449 FY –13.42

LOAD 3 WIND LOAD 0 DEGREE PRESSURE
MEMBER LOAD
58 60 137 139 216 UNI Y –0.79

LOAD 4 WIND LOAD 0 DEGREE SUCTION
MEMBER LOAD
58 60 137 139 216 UNI Y 1.58
```

Similarly for 90 degree pressure and suction loads are applied as member load in the structure.

```
LOAD 7 DEAD + LIVE
REPEAT LOAD
1 1.0 2 1.0
```

Steps:

Load Case 1

To create loads, click on the Load Page icon located on the Structure Tools tool bar.

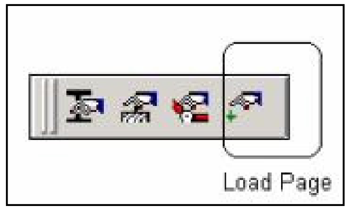


Fig. 5.29

Click on the New Primary button in the Loads dialog box that comes up to initiate the first load case.

Alternatively, one may go to the General | Load Page from the left side of the screen as shown in fig 5.30.

Fig. 5.30

In the Set Active Primary Load Case dialog box that comes up, enter DEAD LOAD as the Title for Load Case 1 and click on OK as shown if fig.5.31.

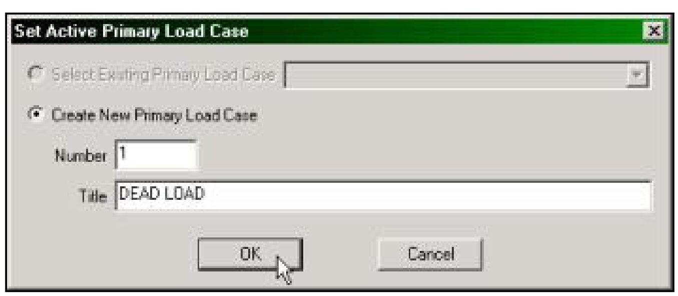


Fig. 5.31

The Loads dialog box will now appear. To generate and assign the selfweight load type, click on the Selfweight button.

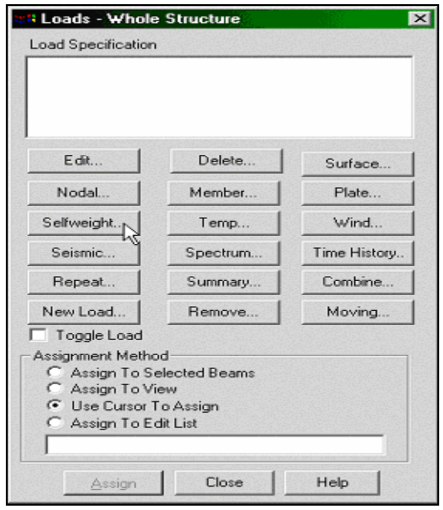


Fig. 5.32

In the Selfweight Load dialog box, specify the Direction as Y, and enter the Factor as -1.0. The negative number signifies that the selfweight load acts opposite to the positive direction of the global axis (Y in this case) along which it is applied. Click on the Assign button. The selfweight load is applicable to every member of the structure, and cannot be applied on a selected list of members.

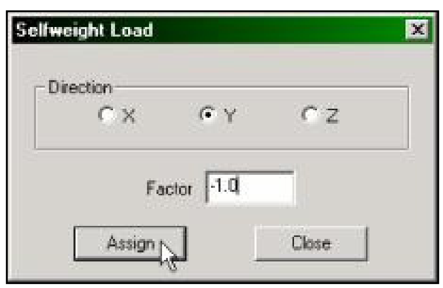


Fig. 5.33

Load 1 contains an additional load component, the joint loads on members 18 and 20. To create this load, click on the Member button in the Loads dialog box.

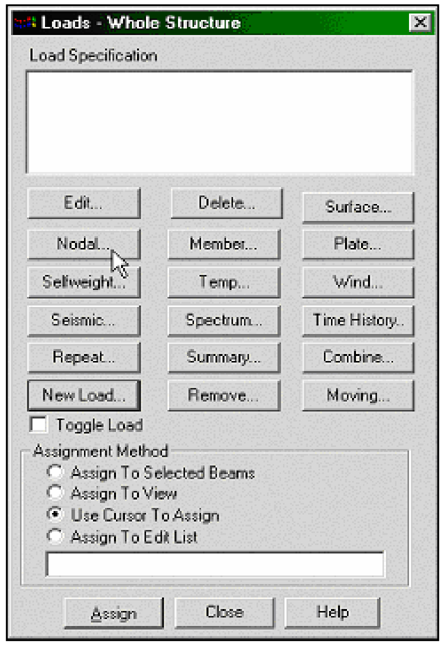


Fig. 5.34

In the Node Loads dialog box that comes up, enter –3.58 for Fy and can straightaway click on the Assign button to apply this load on the selected node.

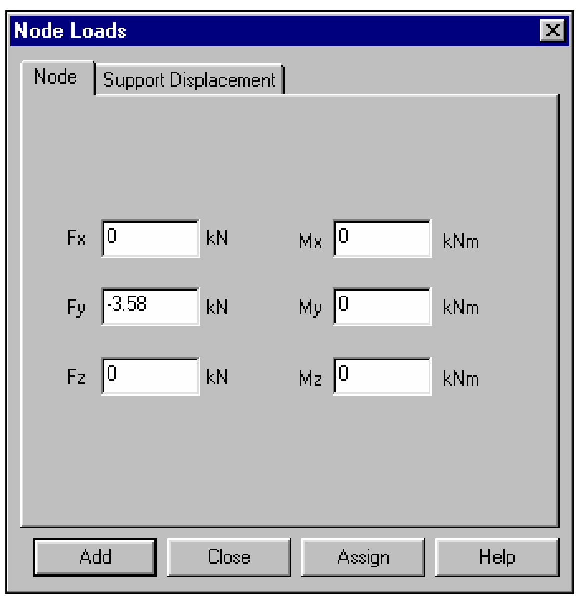


Fig. 5.35

The member load we just created has to be assigned to members 18 and 20. First, make sure that it is selected in the Loads dialog box as shown below.

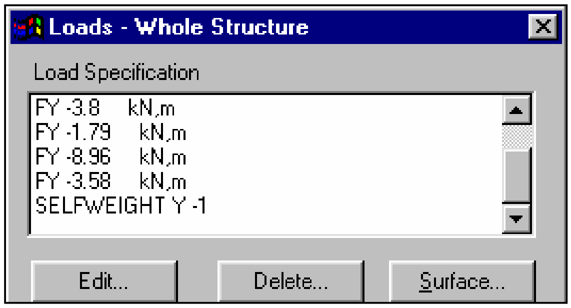


Fig. 5.36

Then, select members 2 and 5 using the Beams Cursor .

Notice that as one should select the members, the Assignment Method automatically becomes Assign to Selected Nodes. Since this is apt for the task, and click on the Assign button in the Loads dialog box.

Fig. 5.37

After the load has been assigned, the structure will look as shown below:

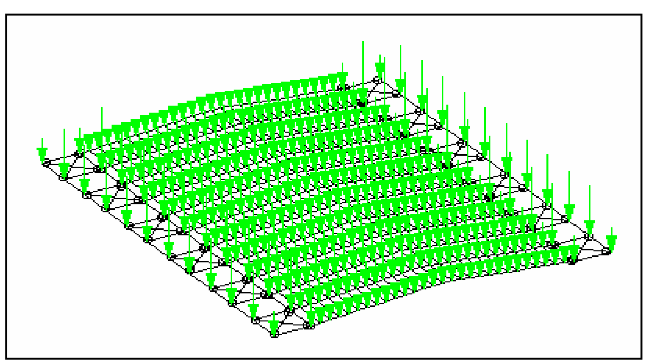


Fig. 5.38

Load Case 2

The next step is to create the second load case, which again contains Joint LOADs. Let the first click on the New Load button in the Loads dialog box to initiate the second load case.

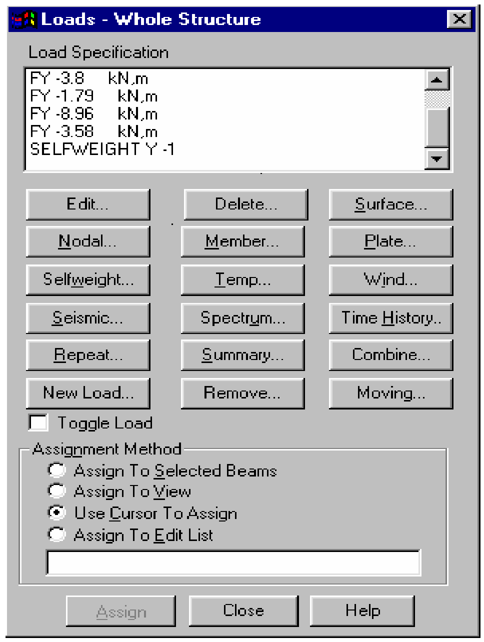


Fig. 5.39

There are two choices available. User is interested in creating a New Primary Load. Let us make that choice and click on the OK button.

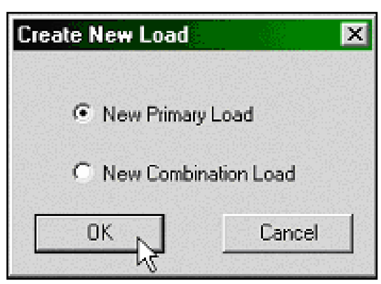


Fig. 5.40

In the New Primary Load dialog box that comes up, let us specify the Title of the second load case as LIVE LOAD and click on the OK button as shown in fig. 5.40.

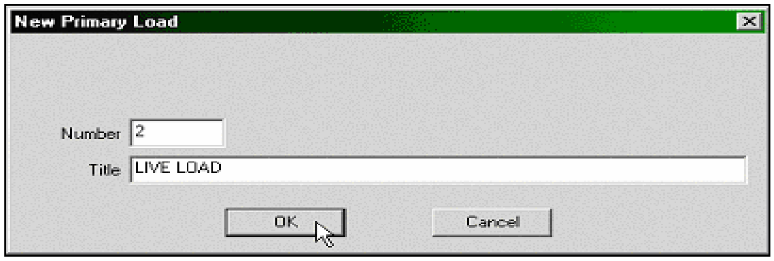


Fig. 5.41

Follow the above steps to create and assign a nodal force of –13.42 KN on members 18 and 20.

Load case 3

Creating the third load case, which now has MEMBER LOADs, involves the procedure as shown below.

As before, first click on the New Primary button in the Loads dialog box to initiate the third load case. In the Set Active Primary Load Case dialog box that comes up, make sure that the Create New Primary Load Case button has been selected. Then, enter WIND LOAD as the Title for Load Case 3 and click on OK.

To apply the load on members follows the procedure as shown below.

In the Beam Loads dialog box that comes up, select the Uniform Force tab. Specify UNI Y as the Direction and enter –0.79 as the Force. For these members, since the local Y-axis coincides with the global Y-axis, one may choose the direction of the load as either "Y" or "GY", they will both have the same effect. (One may view the orientation of the member local axes by going to View | Structure Diagrams | Labels | Beam Orientation.) The negative value signifies that the load acts along the negative UNI Y direction. Then, click on the Add button.

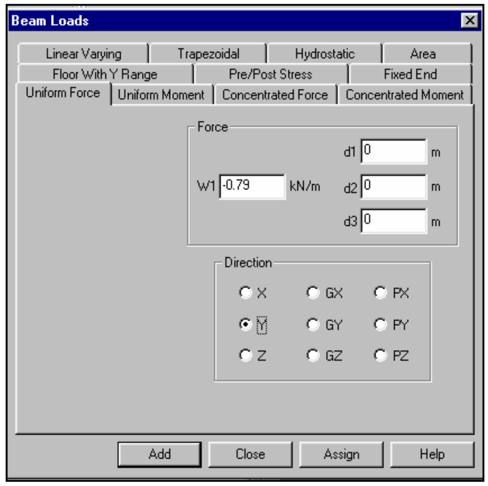


Fig. 5.42

Remaining procedures for all wind loads with suction and pressure for 0 and 90-degree angle are same as for the nodal force to assign the members.

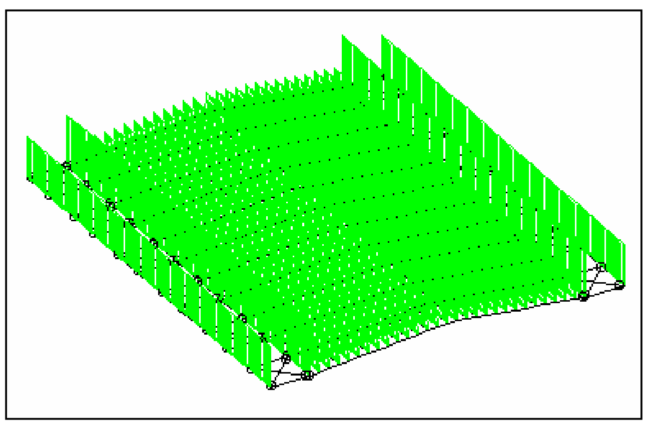


Fig. 5.43

Load case 7

Now come to the point where User has to create load case 7 as (1.0 x Load 1) + (1.0 x Load 2). User saw in the beginning of this section that one should be creating a "REPEAT LOAD" type of combination, and not the "LOAD COMBINATION" type. To initiate load case 7, click on the New Primary button in the Loads dialog box. In the Set Active Primary Load Case dialog box that comes up, make sure that the

Create New Primary Load Case button has been selected. Then, enter DEAD + LIVE as the Title for Load Case 7 and click on OK.

Then, click on the Repeat button in the Loads dialog box as shown below.

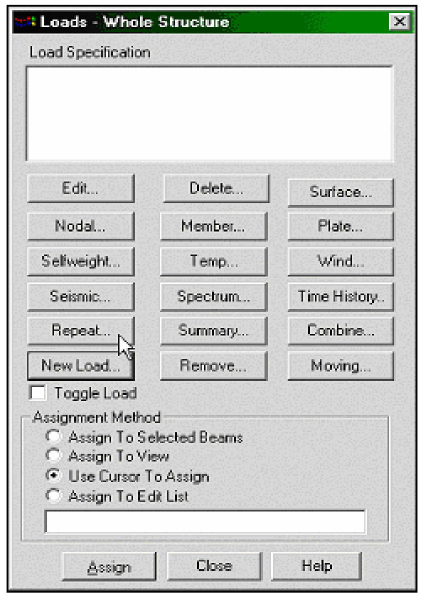


Fig. 5.44

In the Repeat Load dialog box that comes up, select Load Case 1 (DEAD LOAD) and Load Case 2 (LIVE LOAD), enter the Factor as 1.0, and click on the  button. (This indicates that the load data values from load case 1 and case 2 is multiplied by a factor of 1.0, and the resulting values are utilized in load case 7.) Click on the OK button.

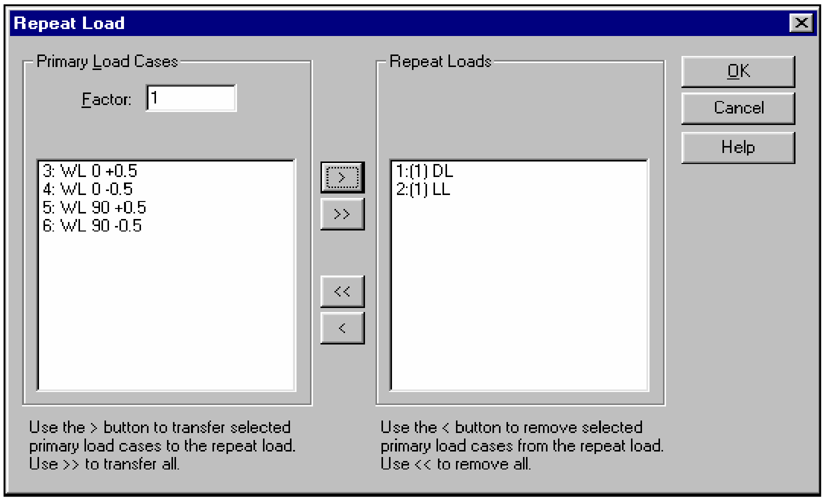


Fig. 5.45

5.2.7 Specifying The Analysis Type

The analysis type for this structure is called Perform Cable Analysis. Since this problem involves geometric nonlinearity for cable elements.

The command for a perform cable analysis will appear in the STAAD file as:

PERFORM CABLE ANALSIS

Go to Analysis/Print Page on the left side of the screen. Then, click on the Analysis sub-page from the second row of pages as shown below.

Fig. 5.46

In the Analysis/Print Commands dialog box that appears, select the Perform Cable Analysis tab. Then, click on the Add button followed by the Close button.

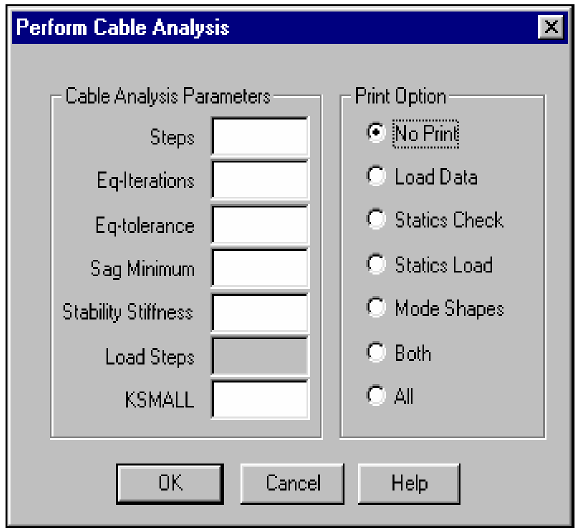


Fig. 5.47

$$\{\text{PERFORM CABLE}\}\quad \text{ANALYSIS}\quad \left(\left[\begin{array}{ll} \text{STEPS} & \text{f1} \\ \text{EQITERATIONS} & \text{f2} \\ \text{EQTOLERANCE} & \text{f3} \\ \text{SAGMINIMUM} & \text{f4} \\ \text{STABILITY} & \text{f5 f6} \\ \text{KSMALL} & \text{f7} \end{array}\right]\right)$$

Steps = Number of load steps. The applied loads will be applied gradually in this many steps. Each step will be iterated to convergence. Default is 15-20. The f1 value, if entered, should be in the range 5 to 145.

Eq-iterations= Maximum number of iterations permitted in each load step. Default is 15 should be in the range of 10 to30.

Eq-tolerance = The convergence tolerance for the above iterations. Default is 0.0005.

Sag minimum=Cables (not trusses) may sag when tension is low. This is accounted for by reducing the E value. Sag minimum may be between 1.0 (no sag E reduction) and 0.0 (full sag E reduction). Default is 1.0. If f4 is entered, it should be in the range 0.7 to 1.0 for a relatively simple process. As soon as SAGMIN becomes less than 0.95 the possibility exists that a converged solution will not be achieved without increasing the steps or the pretension loads. The Eq iterations may need to be 30 or more. The Eq tolerance may need to be greater or smaller.

Stability stiffness = A stiffness matrix value, f5, that is added to the global matrix at each translational direction for joints connected to cables and nonlinear trusses for the first f6 load steps. The amount added linearly decreases with each of the f6 load steps (f6 is 1 if omitted). If f5 entered, use 0.0 to 1000.0. Default is 0.0.

K small stiffness = A stiffness matrix value, f7, that is added to the global matrix at each translational direction for joints connected to cables and nonlinear trusses for every load step. If entered, use 0.0 to 1.0. Default is 0.0.

5.2.8 Viewing The Out Put File

Once the analysis is completed the output file with all analysis results can be seen by clicking on view output file option as shown in fig. 5.48.

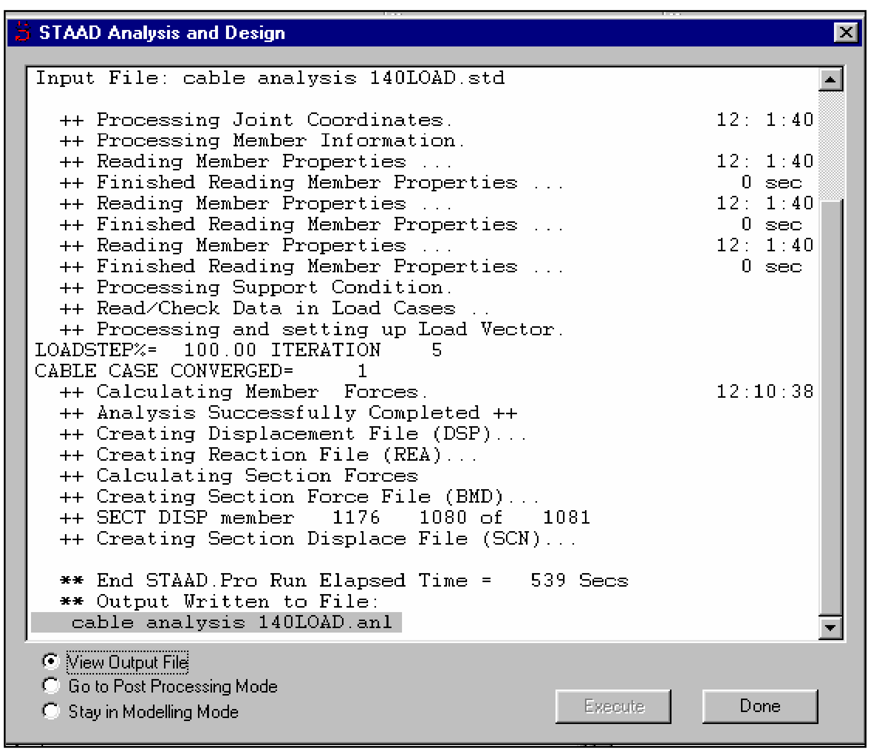


Fig. 5.48

5.3 UNDERSTADNING OF SOFTWARE

STAAD.Pro 2003 is versatile stiffness based software for modeling of any kind of structure. Modeling of cable structure is done by the STAAD.Pro 2003. For the understanding of how to model of a cable in STAAD.Pro 2003, one problem is taken from the book Daniel L. Schodek (Structures Fourth Edition) and performs static linear and compared results of STAAD.Pro with published results.

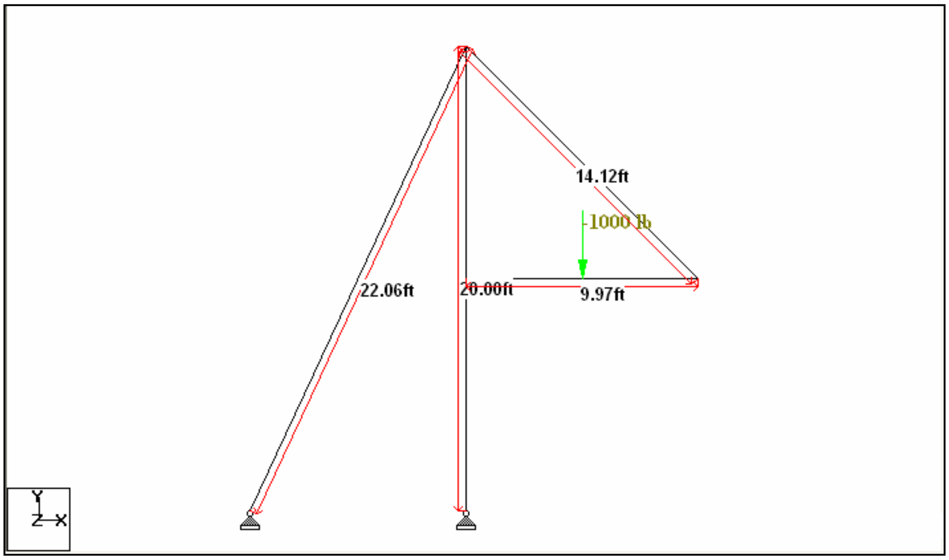


Fig. 5.49 Modeling of Plane Frame

Table 5.1 Comparison of Published and Staad.Pro Results

Element	Published Results	Staad.Pro 2003
Cable Fx **(1) Back** **(2) Fore**	 592 lb 707 lb	 590.680 lb 706.462 lb
Girder Fx	500 lb	498.887 lb
Reaction		
(1) Column	Fx 250 lb Fy 1536 lb	Fx 249.444 lb Fy 1535.825 lb
(2) Cable	Fx -250 lb Fy -536 lb	Fx -249.444 lb Fy -535.425 lb

CHAPTER 6 TYPES OF LOADS AND LOAD COMBINATION

6.1 GENERAL

The loads to be considered in the design of cable roofs are the same as in any other types of roof, apart from the pretension, which cannot strictly be considered as an applied load.
Any error in load assessment will make the roof design erroneous and therefore it makes more important for a designer to calculate the loads of the roofs correctly so that the design becomes economical and safe against the loads. In load calculations the wind plays a vital role. The correct assignment of wind will lead to proper load assessment and reliable design of roof structure.

6.2 DIFFERENT TYPES OF LOADS

6.2.1 Dead Load

The dead load consists of the weight of cladding, insulating material, cables and fittings, etc. The value of dead weight may be as low 240 to 720 N / m^2 if cloth, plastics, or corrugated metal sheets are used and may be anywhere from 720 to 1440 N / m^2 if concrete or timber is used. In the modeling of cable-supported roof, weight of sheeting is used 200 N / m^2.

6.2.2 Live Load

Most specifications specify the intensity of a uniformly distributed live load to be taken into account. In making use of these specifications, however the design should consider that a cable roof generally has a large span and has a curved surface. It can thus be considered inaccessible to people except for maintenance purposes. Therefore, it may be quite justified to design the cladding units for the normal live load, but a lighter design live load may be considered in designing the cable system and the supporting structure. For design purpose live load should be minimum of 750 N / m^2.

6.2.3 Snow Load

In areas where snowfall occurs, the load corresponding to the expected intensity of snow has to be considered. The intensity to be taken into account will vary with the geographic location of the structure. It is worth mentioning that snow accompanied by wind gusts can cause unequal snow intensity over the roof. Similarly, alternate thawing and freezing may lead to nonuniform accumulation. Greater amounts of snow may collect in the flatter parts of the roof. Since unequal loading can result in critical deflections in cable system. When there is a possibility of ponding of melted snow or rainwater, this should be treated as an additional load. Due to climatic region in India no snow load is considered for the analysis of cable supported roof.

6.2.4 Wind Load

Wind forces often govern the design of cable-roof systems. However, there are no special specifications which set out the design wind loads to be considered in designing such systems. It is usual to design cable roofs for a uniform wind pressure or suction, despite the established evidence from experimental studies that this practice is far from satisfactory. Whereas the practice is in many cases unavoidable in view of the lack information on the subject, it is recommended that wherever feasible wind loads on cable-roof surfaces must be determined through wind-tunnel testing. If aero elastic models are considered too expansive, rigid models may be used and will usually afford fairly dependable data. The basic wind pressure depends upon the location of the structure, the values of which can be picked up from Indian Standard Code (IS 875 – Part III) specifications for buildings.

Wind load is considered as time-dependent load. Computations for the static or dynamic effects of wind loads can be accurate as the corresponding estimate of the load, the accuracy of which generally is quite elusive. The general nature of wind is turbulent, but for convenience in analysis, it may be considered to have a steady (or time-averaged) velocity superimposed with a fluctuating component. The fluctuating component, which is caused by the turbulence or gustiness of the wind, leads to a fluctuating pressure and hence a dynamic response.

Calculations of various wind loads in roof and mast or tie-back frame of the structures are discussed in detail:

1) Wind load on roof: The load due to wind on roof surface and facia are applied as a pressure or suction on full span and half pressure on one span and suction on the other half. These wind loads are applied as a nodal load.

   Variations of hourly mean wind speed with height shall be calculated as follows:

$$V_z = V_b \times k_1 \times k_2 \times k_3 \qquad \text{…….. (6.1)}$$

Where

V_z = Hourly mean wind speed in m/s at height z

V_b = Regional basic wind speed in m/s

k_1 = Probability factor

k_2 = Terrain and height factor

k_3 = Topography factor

For the pressure and suction coefficient Table 8 (IS 875 – III) (Pressure coefficients for free standing doubled sloped roofs) are consider. For the cable supported roof pressure coefficient with 5 degree roof angle and solidity ratio ϕ=1 is +0.3 pressure and -0.6 suction is to be consider.

$$F_{roof} = C_p \times A_e \times p_z \times G \qquad \text{…….. (6.2)}$$

Where,

F_{roof} = Wind load at roof surface in KN

C_p = Wind pressure coefficient

A_e = Effective frontal area considered for the structure at height z

p_z = Design pressure at height z due to hourly mean wind obtained as 0.6 V_z^2 (KN/m^2)

G = Gust factor (= peak load / mean load)

$$G = 1 + g_f \; r \sqrt{\left[B\,(1+\phi)^2 + \frac{SE}{\beta} \right]} \qquad \text{…….. (6.3)}$$

Where,

g_f = Peak factor defined as the ratio of the expected peak value to the roof mean value of a fluctuating load

r = Roughness factor which is dependent on the size of the structure in relation to the ground roughness

The value of $g_f\ r$ is given in Fig. 6.1

B = Background factor indicating a measure of slowly varying component of fluctuating wind load and is obtained from Fig. 6.2

$\frac{SE}{\beta}$ = Measure of the resonant component of the fluctuating wind load,

S = Size reduction factor (Fig. 6.3)

E = Measure of available energy in the wind stream at the natural frequency of the structure (Fig. 6.4)

β = Damping coefficient (as a fraction of critical damping) of the structure

= 0.010 for welded structure

ϕ = $\frac{g_f\ r\sqrt{B}}{4}$ and is to be accounted only for building less than 75 m high in terrain category and building less than 25 m high in terrain category 3, and is to be taken as zero in all other cases.

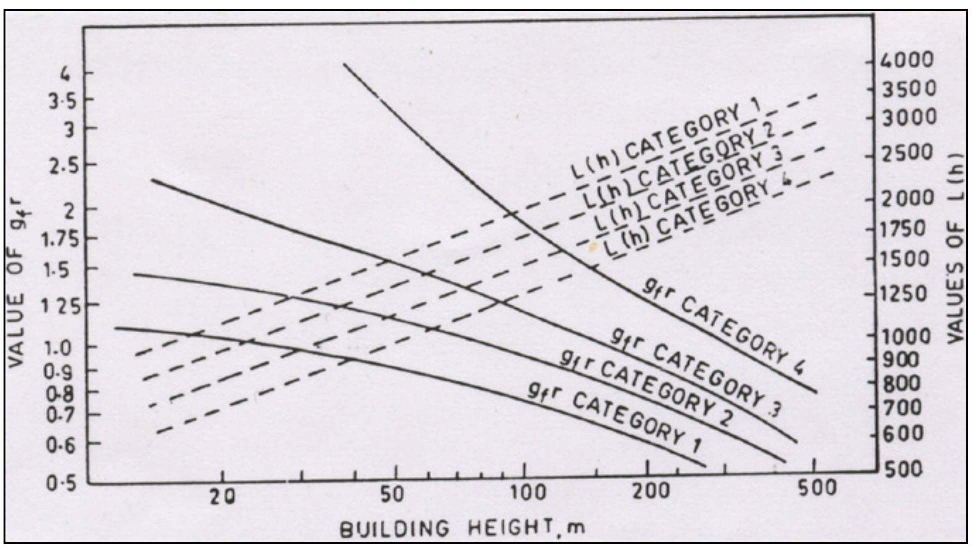


Fig. 6.1 Values of $g_f r$ and L (h)

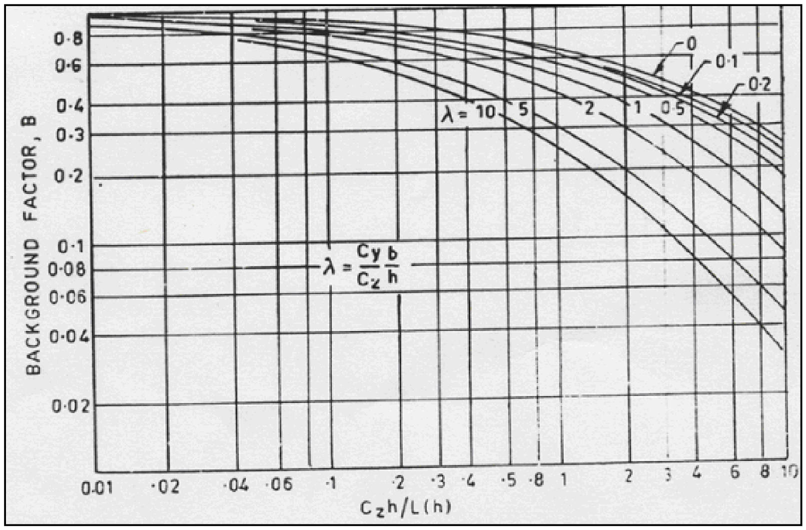


Fig. 6.2 Background Factor B

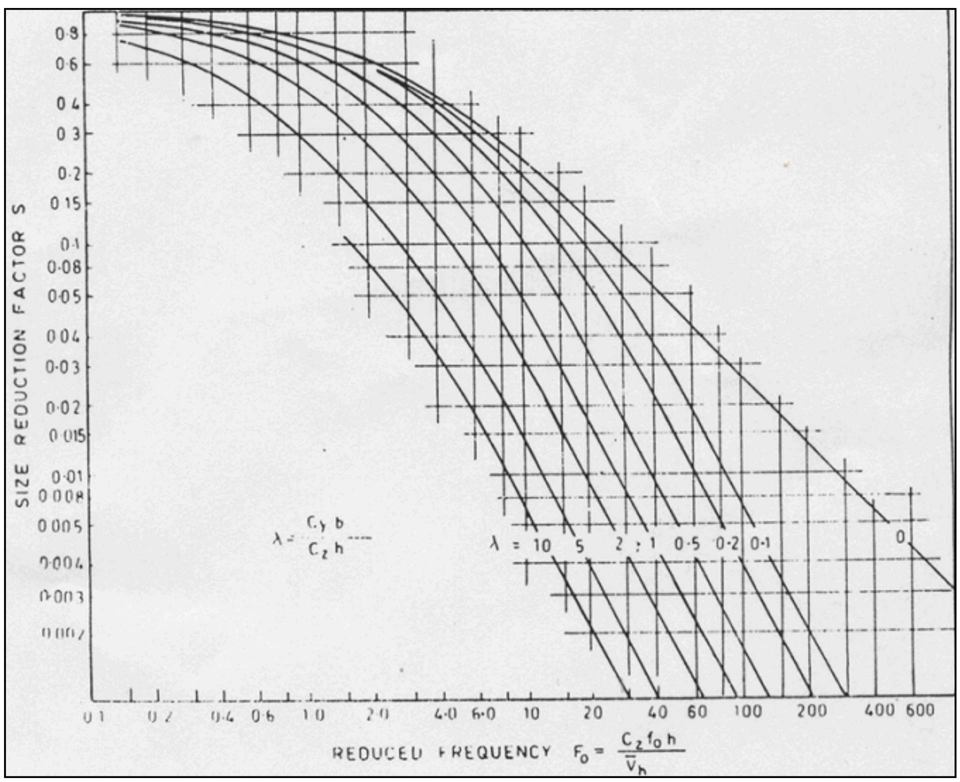


Fig. 6.3 Size Reduction Factor, S

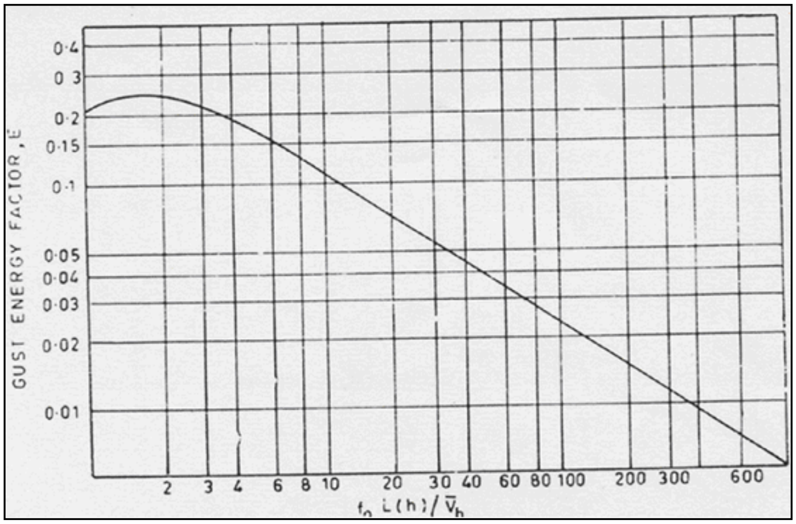


Fig. 6.4 Gust Energy Factor, E

In fig. 6.1 to 6.4

$$\lambda = \frac{C_y b}{C_z h} \qquad \text{........ (6.4)}$$

$$f_0 = \frac{C_z f_o h}{V_h} \qquad \text{........ (6.5)}$$

Where

C_y = Lateral correlation constant which may be taken as 10 in the absence of more precise load data

C_z = Longitudinal correlation constant which may be taken as 12 in the absence of more precise load data

b = Breadth of a structure normal to the wind stream

h = Height of a structure

V_h = Hourly mean wind speed at height z

f_0 = Natural frequency of the structure

$L(h)$ = A measure of turbulence length scale as shown in fig. 6.2

2) Wind load on cable:

Similarly for Cables,

$$F_{cable} = C_{fc} \times l \times d \times p_z \times G \qquad \text{........ (6.6)}$$

Where,

F_{cable} = Wind load on cable member in KN

C_{fc} = Force Coefficient for wires and cables ($DV_d > 6\ m^2/s$),

l = Length of Cable member in m,

d = Diameter of cable member in m,

p_z = Design pressure at height z due to hourly mean wind obtained as 0.6 V_z^2 (kN/m^2),

G = Gust factor (= peak load / mean load)

3) Wind load on mast:

$$F_{mast} = C_f \times A_e \times p_z \times G \qquad \text{........ (6.7)}$$

Where,

F_{mast} = Along wind load on particular member in KN,

C_f = Force coefficient for the structure,

A_e = Effective frontal area considered for the structure at height z,

p_z = Design pressure at height z due to hourly mean wind obtained as 0.6 V_z^2 (KN/m^2),

G = Gust factor (= peak load / mean load)

6.2.5 Earthquake load

An earthquake is a ground motion which causes the entire cable-roof system is vibrate along with the supporting structure. The response spectrum method is a very popular analysis tool for predicting the response of complex structural systems to earthquake ground motions. The general procedure is to compute the response of each of the individual modes of the structures and then to combine theses responses to obtain overall response. In many cases, only a few of the modes must be included when computing any particular response of the system. The specific modes, which must be considered, will depend upon the properties of the structure and the particular quantity, which is being computed. The computations, which are required, are usually less than those required for a time history analysis.

Response Spectrum is a dynamic method of analysis. In calculation of structural response (whether model analysis or otherwise), the structure should be so represented by means of an analytical or computational model that reasonable and rational results can be obtained by its behaviour. Where response spectrum method is used with model analysis procedure, at least three modes of response of the structure should be considered except in those cases where it can be shown qualitatively that either third mode or the second mode produces negligible response. When appropriate, the model maxima should be combined using the square root of the sum of the squares of the individual model values. In this method the structure is considered as a flexible structure with lumped masses concentrated at floor levels, with each mass having one degree of freedom that is of lateral displacement in the direction under consideration.

In response spectrum method the response acceleration coefficient is first obtained for the natural period and the damping of the structure and the design value of horizontal seismic coefficient is computed using the following expression:

$$V_B = A_h \times W \qquad \text{....... (6.8)}$$

$$A_h = \frac{Z \times I \times Sa}{2 \times R \times g} \qquad \text{....... (6.9)}$$

Z = 0.16 Zone Factor for zone III of moderate seismic intensity

I = 1 Importance factor for all buildings and structures

R = Response reduction factor

= 5 for Special RC Moment Resisting Frame (SMRF)

= 4 for Steel Frame with Concentric Braces

= 5 for Steel Frame with Eccentric Braces

= 3 for Ordinary Moment Resisting Frame (OMRF)

$\frac{Sa}{g}$ = Average Response Acceleration coefficient

W = Seismic weight of the building

Table 6.1 Time periods Vs Ah

	Time Period	Acceleration	Ah
	Ta	**Sa/g**	**Steel**
1	0	1.00	0.028
2	0.1	2.50	0.070
3	0.2	2.50	0.070
4	0.3	2.50	0.070
5	0.4	2.50	0.070
6	0.5	2.50	0.070
7	0.6	2.27	0.063
8	0.7	1.94	0.054
9	0.8	1.70	0.048
10	0.9	1.51	0.042
11	1	1.36	0.038
12	1.1	1.24	0.035
13	1.2	1.13	0.032
14	1.3	1.05	0.029
15	1.4	0.97	0.027
16	1.5	0.91	0.025
17	1.6	0.85	0.024
18	1.7	0.80	0.022
19	1.8	0.76	0.021
20	1.9	0.72	0.020
21	2	0.68	0.019
22	2.1	0.65	0.018
23	2.2	0.62	0.017
24	2.3	0.59	0.017
25	2.4	0.57	0.016
26	2.5	0.54	0.015
27	2.6	0.52	0.015
28	2.7	0.50	0.014
29	2.8	0.49	0.014
30	2.9	0.47	0.013
31	3	0.45	0.013
32	3.1	0.44	0.012
33	3.2	0.43	0.012
34	3.3	0.41	0.012
35	3.4	0.40	0.011
36	3.5	0.39	0.011
37	3.6	0.38	0.011
38	3.7	0.37	0.010
39	3.8	0.36	0.010
40	3.9	0.35	0.010
41	4	0.34	0.010

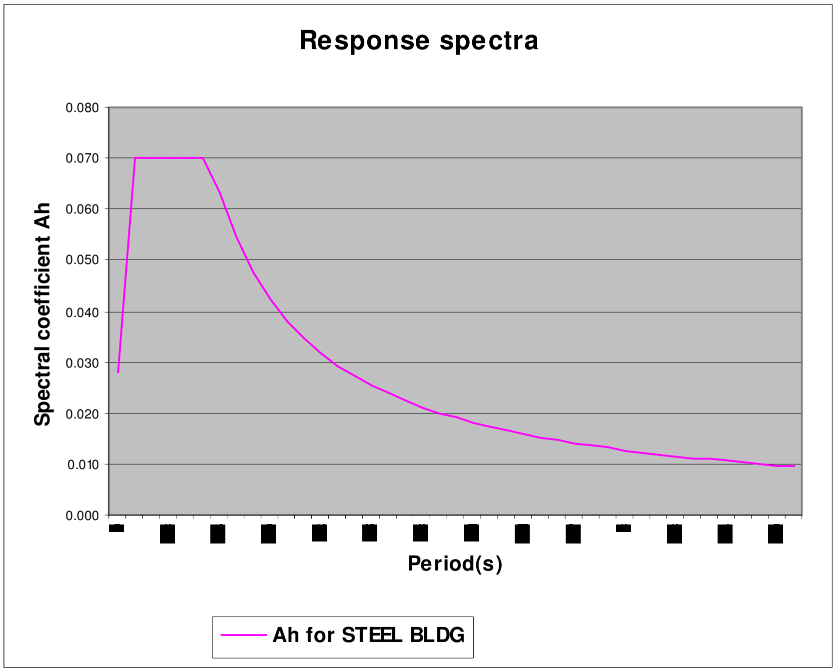


Fig.6.5 Response spectra for cable roof

6.2.6 Pretension Load

Prestress can be treated as a permanent load similar to dead load. Pretensioning is achieved either by preloading the cables, or by employing counter stressed systems. To control deformation in a cable net, the cable system is pre-tensioned. This prestension is designed to elevate the cable behaviour above the range in which the sag deformation become significant. Usually, cables are prestretched to 4-10% of their minimum breaking strength, depending on their size.

6.2.7 Erection Load

Loads in which the structure is subjected during erection are temporary but can be quite heavy. Erection loads and their effect need to be worked out at every important stage during construction of roof. Erection stresses can be severe in supporting structures likes rings unless the prestressing procedure is carefully planed.

6.2.8 Creep and Temperature Effects

In addition to the above-mentioned loads, the effect of creep in the supporting structure in the cables has to be taken into account. In both areas creep will result in relaxing the cable tension and thus reducing the stiffness of the system. The effect of temperature rise or fall in a cable system can be incorporated in to the following way.

$$F_i^t = E \times Ai \times \alpha \times t \qquad \text{.......(6.10)}$$

Where,

α = Coefficient of linear expansion

t = Change in temperature

F_i^t = Change in force due to temperature in a typical element i

6.3 LOAD CASES CONSIDERED FOR WIND LOAD (GUST FACTOR)

Load case 1.	Dead Load (Self weight of roof components + roofing sheet)
Load case 2.	Live Load on Full Span
Load case 3.	Wind Load in X Direction (0 Degree with Pressure)
Load case 4.	Wind Load in X Direction (0 Degree with Suction)
Load case 5.	Wind Load in X Direction (0 Degree with Half Pressure and Half Suction)
Load case 6.	Wind Load in Z Direction (90 Degree with Pressure)
Load case 7.	Wind Load in Z Direction (90 Degree with Suction)
Load case 8.	Wind Load in Z Direction (90 Degree with Half Pressure and Half Suction)
Load case 9.	D.L. + L.L. + 0 Degree Pressure Wind Load in X Direction
Load case 10.	D.L. + L.L. - 0 Degree Suction Wind Load in X Direction
Load case 11.	D.L. + L.L. $\pm$ 0 Degree Half Pressure and Suction Wind Load in X Direction
Load case 12.	D.L. + L.L. + 90 Degree Pressure Wind Load in Z Direction

Load case 13.	D.L. + L.L. - 90 Degree Suction Wind Load in Z Direction
Load case 14.	D.L. + L.L. ± 90 Degree Half Pressure and Suction Wind Load in Z Direction
Load case 15.	0.9 (D.L. + L.L.) + 0 Degree Pressure Wind Load in X Direction
Load case 16.	0.9 (D.L. + L.L.) - 0 Degree Suction Wind Load in X Direction
Load case 17.	0.9 (D.L. + L.L.) ± 0 Degree Half Pressure and Suction Wind load in X Direction
Load case 18.	0.9 (D.L. + L.L.) + 0 Degree Pressure Wind Load in Z Direction
Load case 19.	0.9 (D.L. + L.L.) - 0 Degree Suction Wind Load in Z Direction
Load case 20.	0.9 (D.L. + L.L.) ± 0 Degree Half Pressure and Suction Wind load in Z Direction
Load case 21.	D.L. + 0 Degree Pressure Wind Load in X Direction
Load case 22.	D.L. - 0 Degree Suction Wind Load in X Direction
Load case 23.	D.L. ± 0 Degree Half Pressure and Suction Wind Load in X Direction
Load case 24.	D.L. + 0 Degree Pressure Wind Load in Z Direction
Load case 25.	D.L. - 0 Degree Suction Wind Load in Z Direction
Load case 26.	D.L. ± 0 Degree Half Pressure and Suction Wind Load in Z Direction

6.4 LOAD CASES CONSIDERED FOR SEISMIC FORCES (RESPONSE SPECTRUM METHOD)

Load case 27.	Earthquake Force in X Direction
Load case 28.	Earthquake Force in Z Direction
Load case 29.	D.L. + L.L. + EQ in +X Direction
Load case 30.	D.L. + L.L. + EQ in -X Direction
Load case 31.	D.L. + L.L. + EQ in +Z Direction
Load case 32.	D.L. + L.L. + EQ in -Z Direction
Load case 33.	0.9(D.L. + L.L.) + EQ in +X Direction
Load case 34.	0.9(D.L. + L.L.) + EQ in -X Direction
Load case 35.	0.9(D.L. + L.L.) + EQ in +Z Direction
Load case 36.	0.9(D.L. + L.L.) + EQ in -Z Direction
Load case 37.	D.L. + EQ in +X Direction
Load case 38.	D.L. + EQ in -X Direction
Load case 39.	D.L. + EQ in +Z Direction

Load case 40. D.L. + EQ in -Z Direction

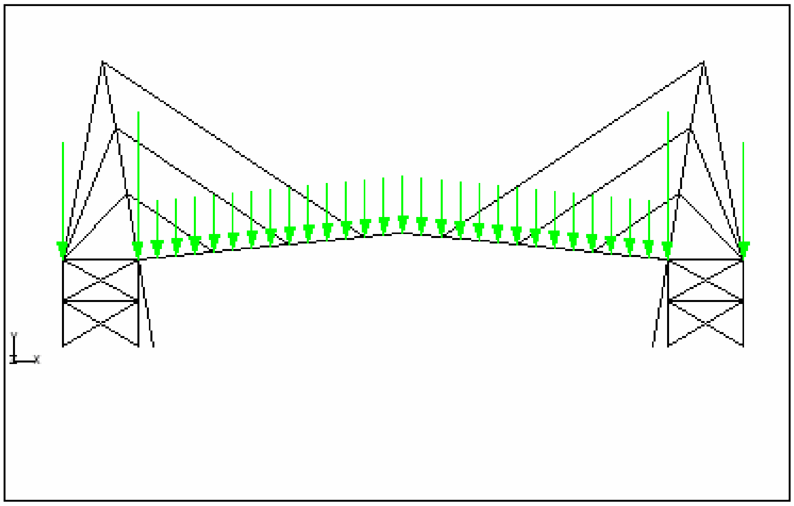


Fig. 6.6 Dead Load

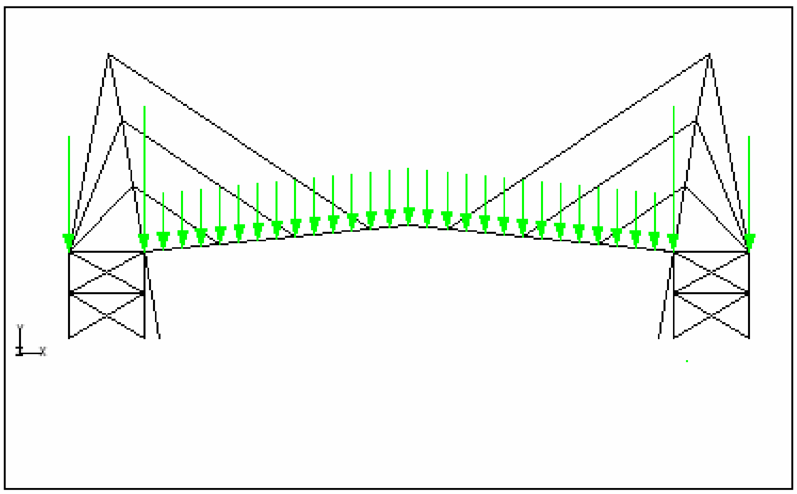


Fig. 6.7 Live Load

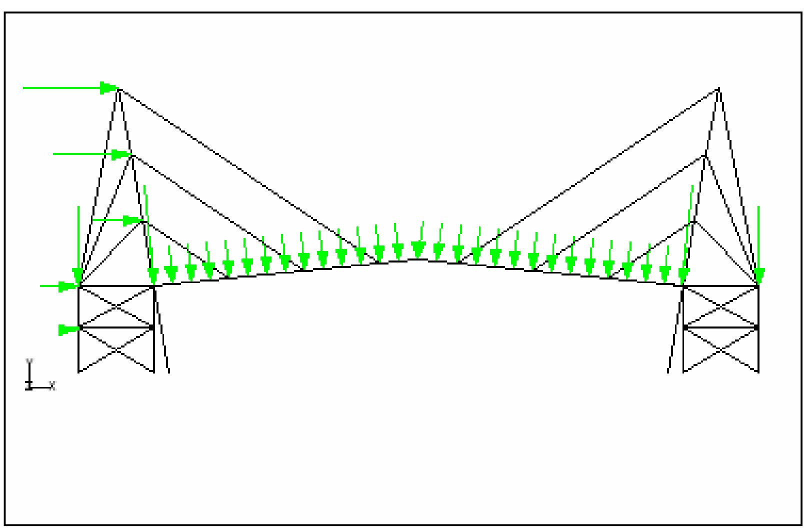


Fig. 6.8 Wind Load (Pressure) in X Direction

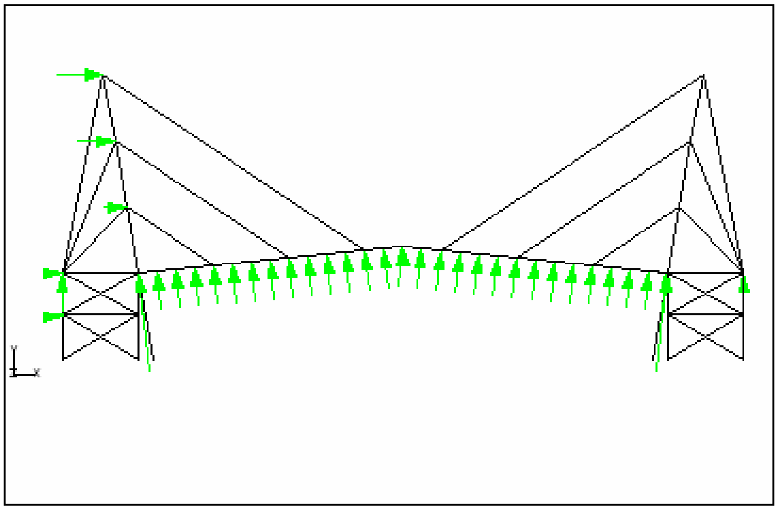


Fig. 6.9 Wind Load (Suction) in X Direction

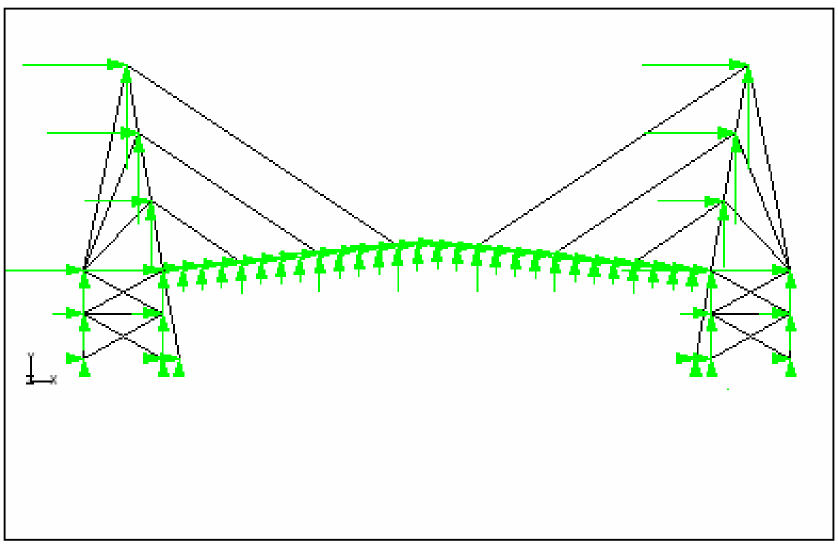


Fig. 6.10 Earthquake in X direction

CHAPTER 7 STATIC LINEAR ANALYSIS RESULT

The analysis results obtained after linear static analysis with different load cases using STAAD.Pro are listed in tabulated form in the following pages.

The results of Maximum Cable Force Fx in kN, Maximum Mast Force in kN, Maximum Mast Moment in kN-m, Maximum Main Girder Force in kN, Maximum Main Girder Moment in kN-m, Maximum Purlin Force in kN and Maximum Purlin Moment in kN-m, Maximum Tie-back Frame Force in kN and Max Tie-back Frmae moment in kN-m are sorted from the analysis results obtained from the STAAD.Pro.

From studying analysis results we can see that for most of all component parts of the structure the dead load + live load + wind load in x direction is the ruling force combination and for that basic design is prepared.

Static linear analysis results are discussed for the following components.

- Cable Force
- Main Girder Force
- Main Girder Moment
- Mast Force
- Mast Moment
- Purlin Force
- Purlin Moment
- Tie-back Column Force
- Tie-back Column Moment

Table 7.1 Results of Static Linear Analysis for Cables

	Load Case 9	Load Case 10	Load Case 14	Load Case 29
Max Tension (Fx) kN	390.61	476.51	446.10	394.57

Table 7.2 Results of Static Linear Analysis for Main Girder

	Load Case 9	Load Case 10	Load Case 14	Load Case 29
Max Axial Force (Fx) kN	632.13	765.00	623.92	635.37
Max Bending Moment (Mz) kN-m	272.11	322.95	290.01	269.13

Table 7.3 Results of Static Linear Analysis for Purlin

	Load Case 9	Load Case 10	Load Case 14	Load Case 29
Max Axial Force (Fx) kN	8.98	10.81	8.38	9.00
Max Bending Moment (Mz) kN-m	19.14	23.76	21.27	18.45

Table 7.4 Results of Static Linear Analysis for Mast

	Load Case 9	Load Case 10	Load Case 14	Load Case 29
Max Axial Force (Fx) kN	2396.43	2894.59	2645.20	2403.33
Max Bending Moment (Mz) kN-m	172.97	209.16	205.28	172.96

Table 7.5 Results of Static Linear Analysis for Tie-Back Column

	Load Case 9	Load Case 10	Load Case 14	Load Case 29
Max Tension (Fx) kN	715.72	874.84	785.08	721.65
Max Bending Moment (Mz) kN-m	138.77	164.81	144.79	134.38

Note:

Load case 9	Dead Load + Live Load
Load case 10	Dead Load + Live Load + Wind Load (pressure) in X Direction
Load case 14	Dead Load + Live Load + Wind Load (Half Pressure and Suction in X Direction
Load case 29	Dead Load + Live Load + Earthquake in X direction

Table 7.6 Deflection of Linear Static Analysis

NODE	X (mm)	Y (mm)	Z (mm)
692	1.52	-340.82	-0.15
495	29.19	-7.65	0.88
244	-19.40	-119.85	3.17
496	114.49	0.60	-0.27
245	-8.55	-236.54	2.69
497	196.38	11.93	-2.12
246	0.11	-326.69	1.29

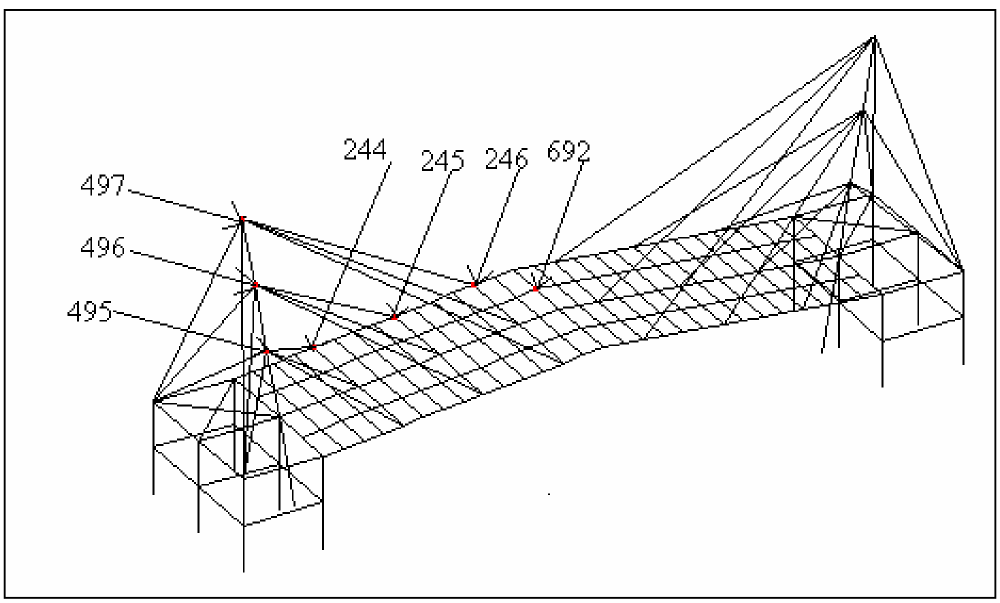


Fig. 7.1 Deflected Node Number

Table 7.7 Support Reactions

	Horizontal	Horizontal	Vertical	Horizontal	Moment		
Node	L/C	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
22	12	337.47	-592.73	-0.94	-6.03	0.00	-61.13
452	10	-343.15	-603.91	-0.13	1.19	0.00	62.29
296	10	-29.67	2056.65	-0.35	0.60	0.00	148.07
136	10	25.43	-813.99	0.26	-0.48	0.00	-125.71
493	10	-110.88	787.09	3.37	18.17	3.21	-135.47
486	12	105.11	765.26	-4.45	-23.97	4.23	145.81
493	10	-110.88	787.09	3.37	18.17	3.21	-135.47
486	12	105.11	765.26	-4.45	-23.97	4.23	145.81
486	12	105.11	765.26	-4.45	-23.97	4.23	145.81
477	25	-19.77	124.11	-0.80	-4.15	-0.73	-7.11
294	10	-31.28	87.60	0.12	1.08	0.00	149.99
175	12	30.79	86.00	-1.3	-6.08	0.00	-146.72

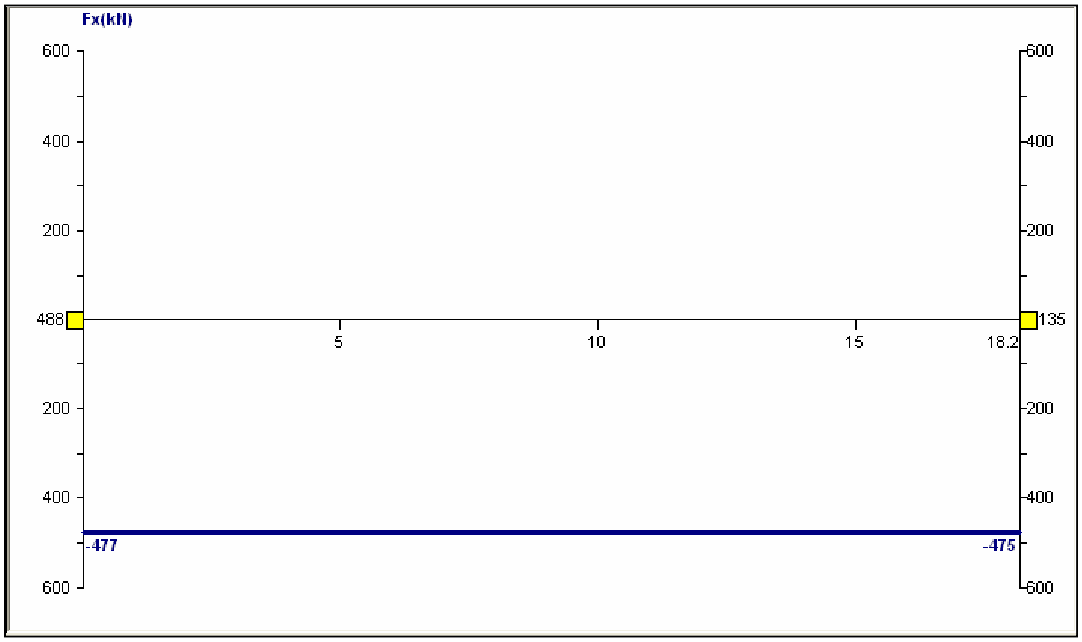


Fig. 7.2 Cable (Axial Force)

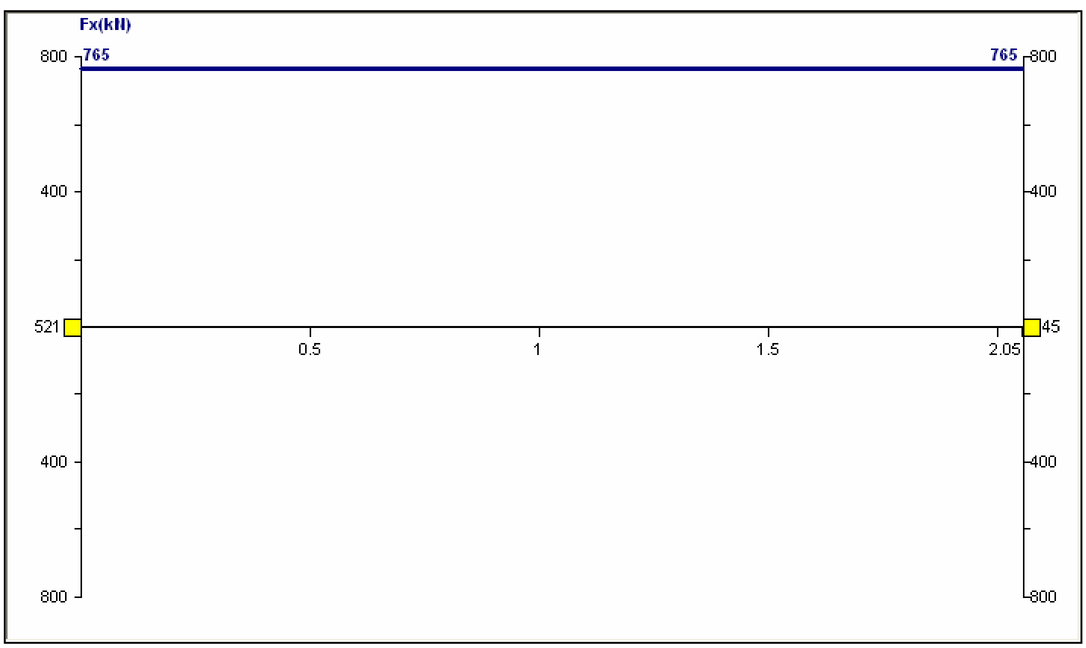


Fig. 7.3 Main Beam (Axial Force)

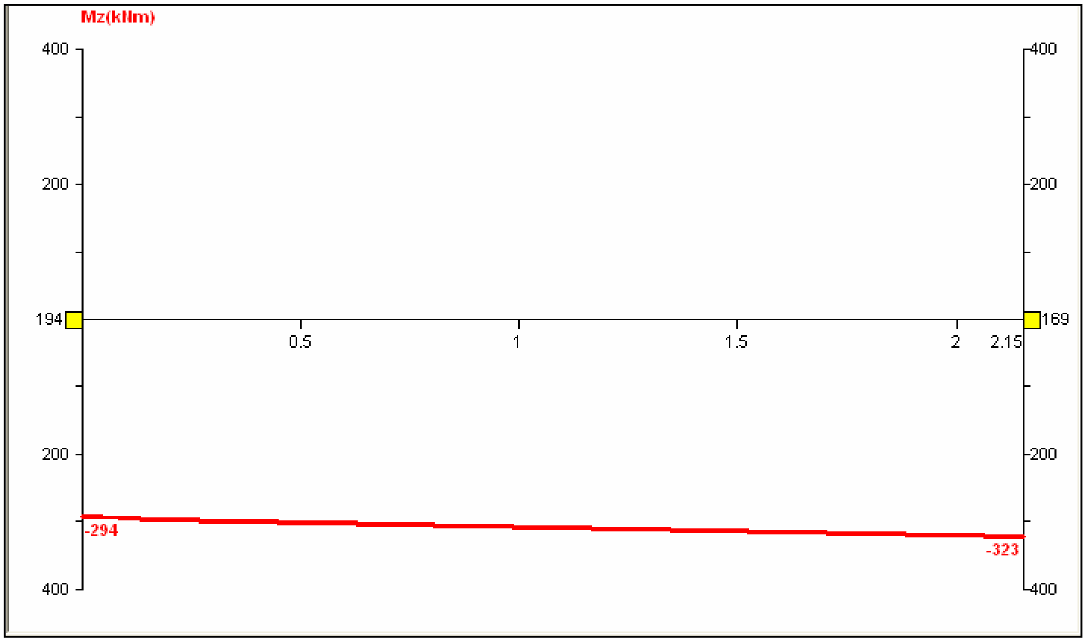


Fig. 7.4 Main Beam (Bending Moment)

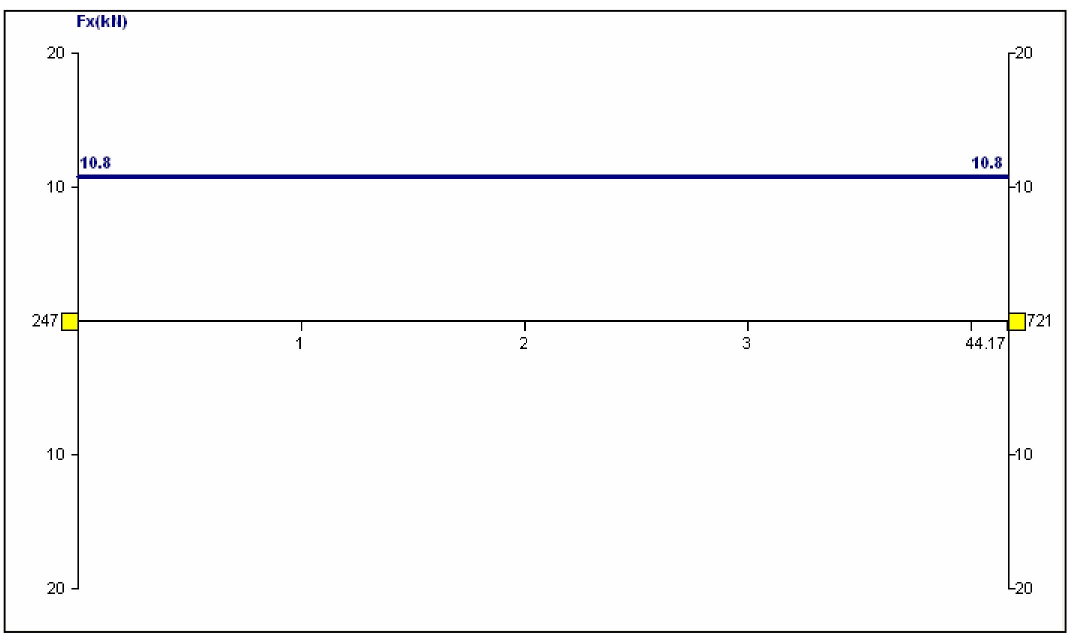


Fig. 7.5 Purlin (Axial Force)

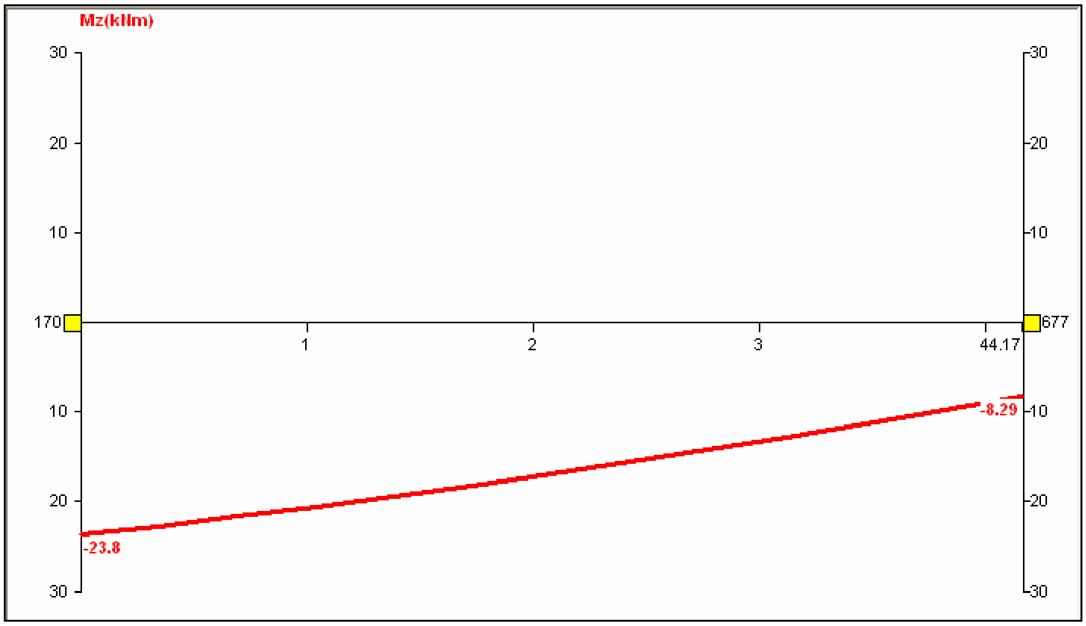


Fig. 7.6 Purlin (Bending Moment)

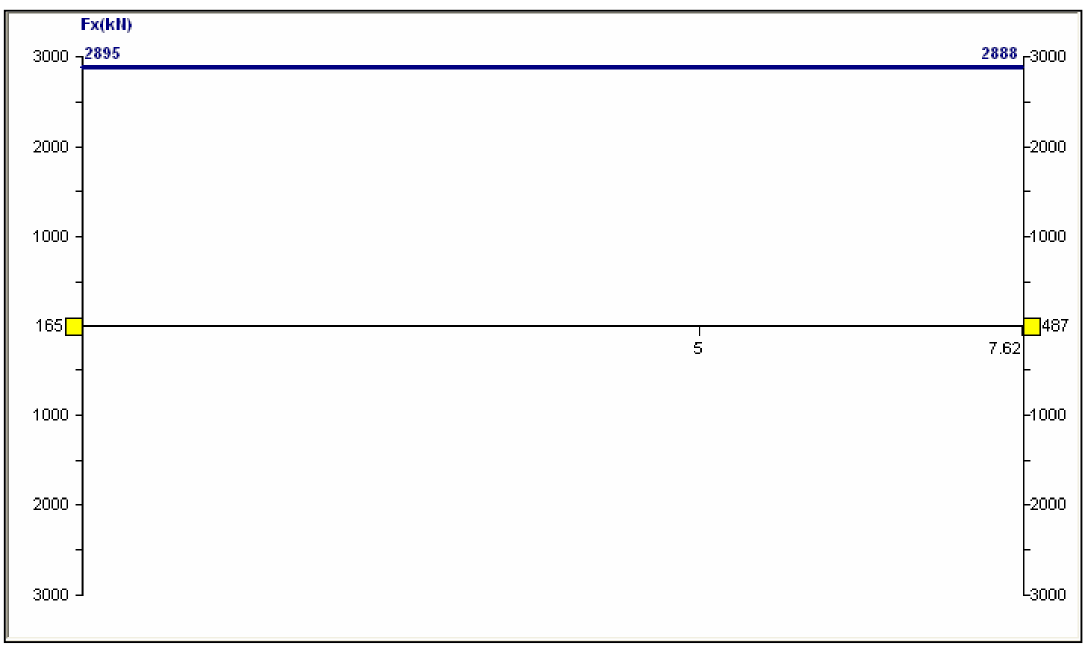


Fig. 7.7 Mast (Axial Force)

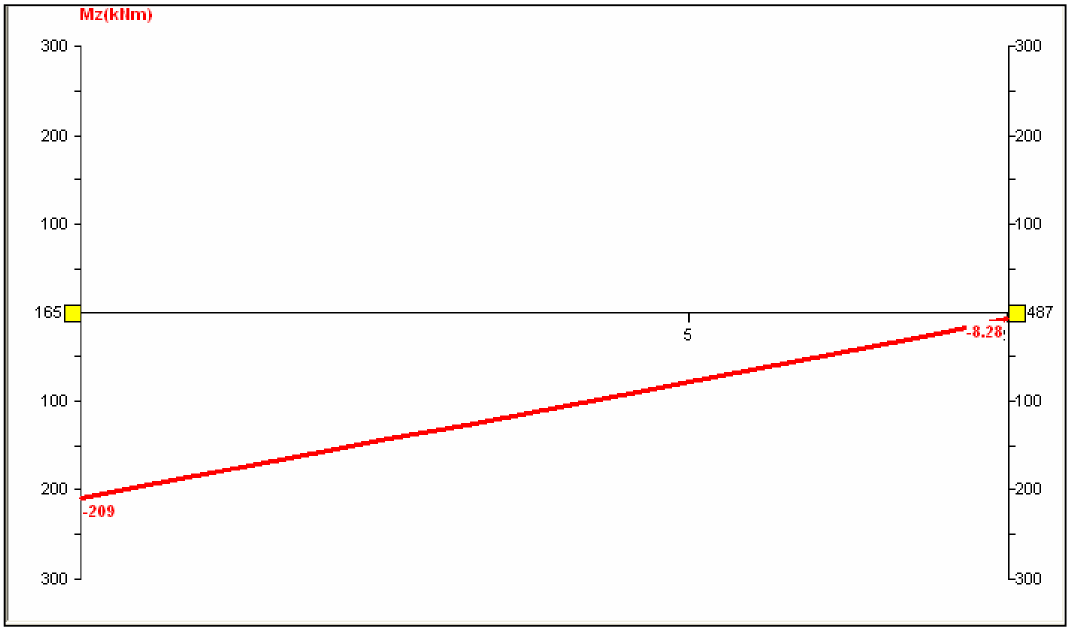


Fig. 7.8 Mast (Bending Moment)

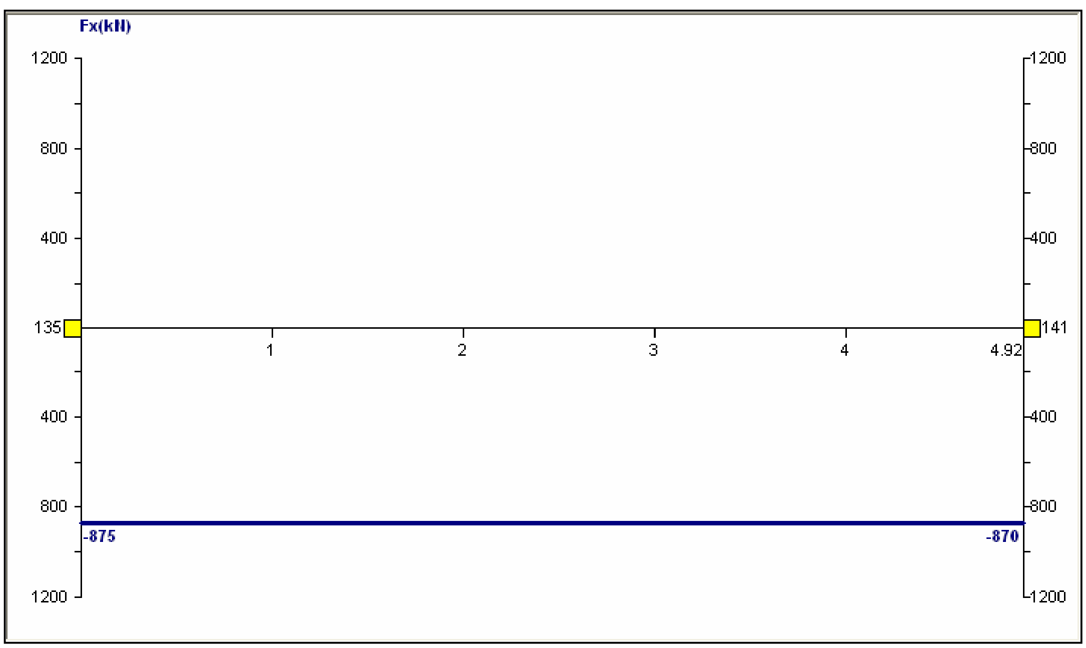


Fig. 7.9 Tie-Back Frame (Axial Force)

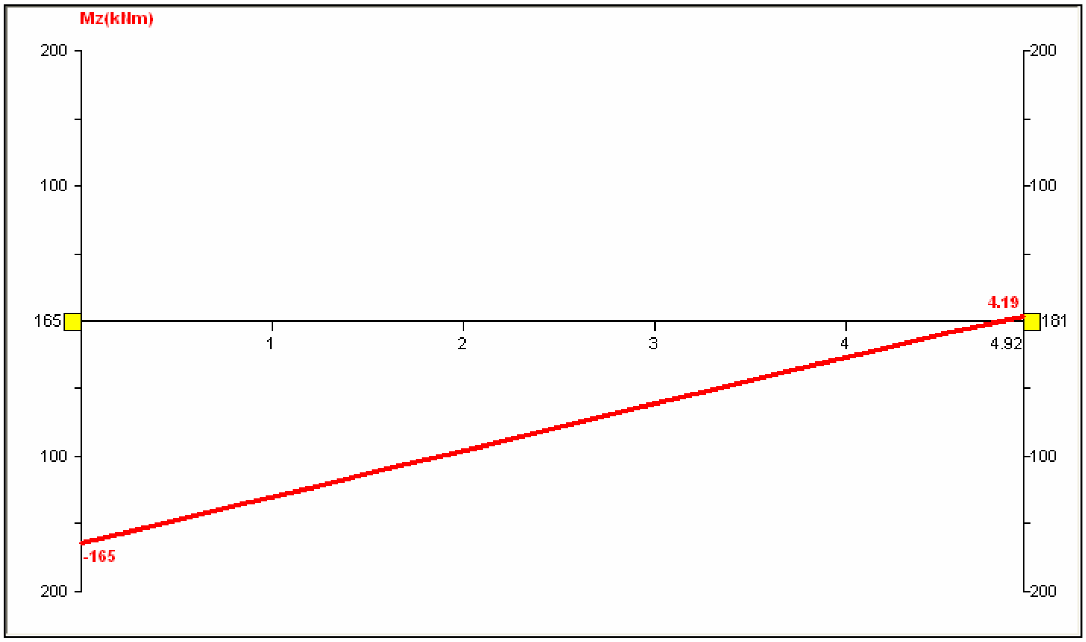


Fig. 7.10 Tie-Back Frame (Bending Moment)

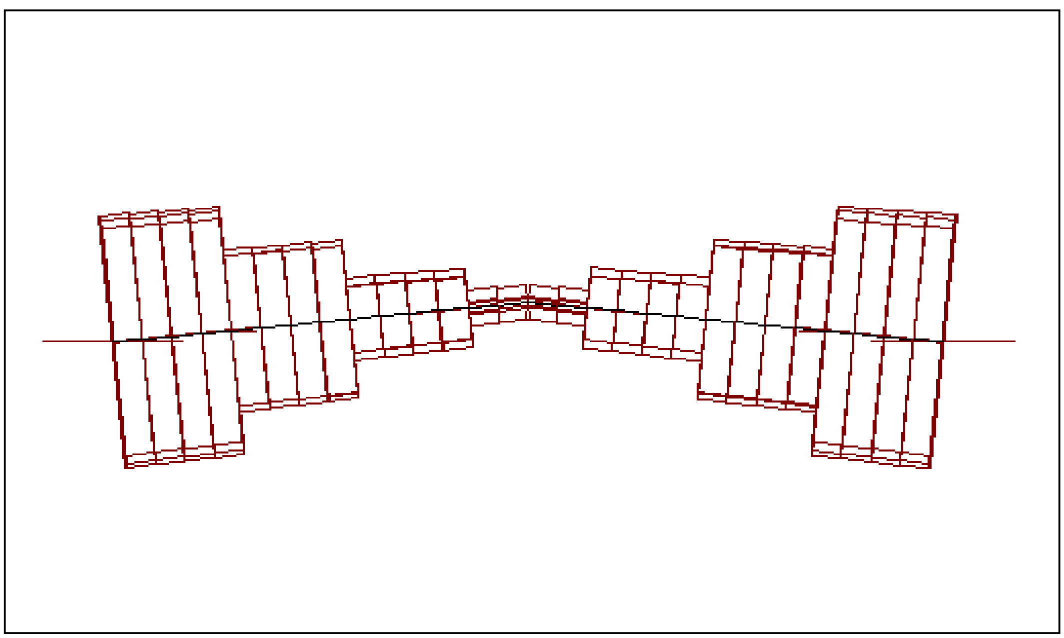


Fig. 7.11 Whole Structure Beam (Axial Force)

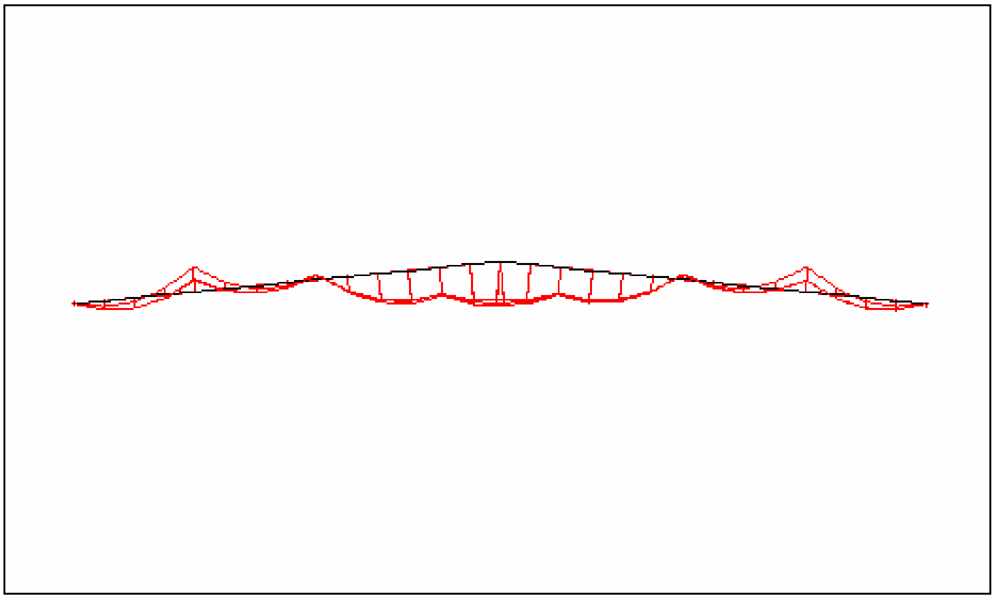


Fig. 7.12 Whole Structure Beam (Bending Moment)

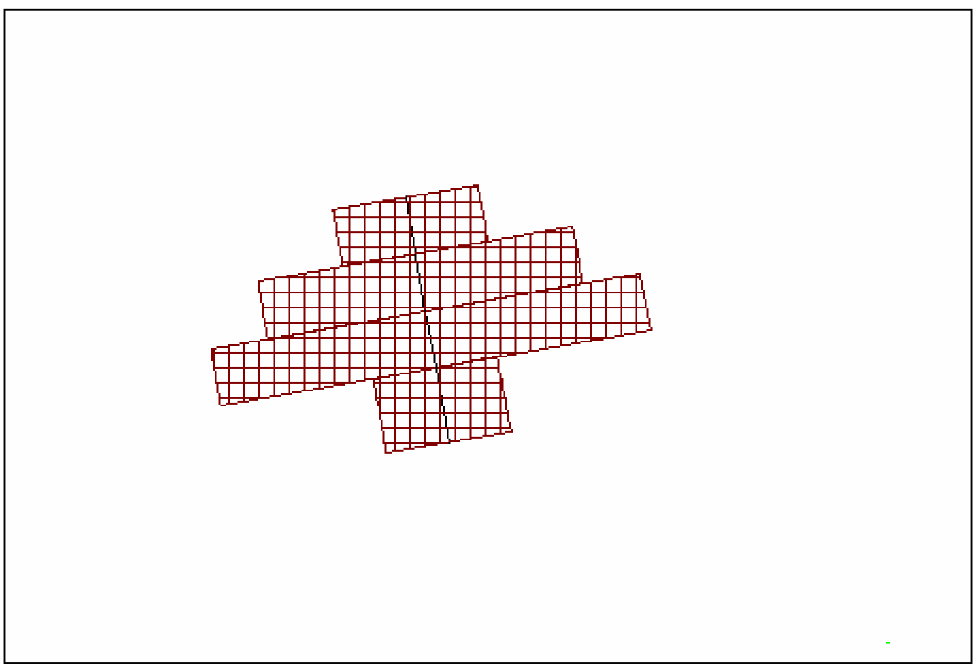


Fig. 7.13 Whole Structure Mast (Axial Force)

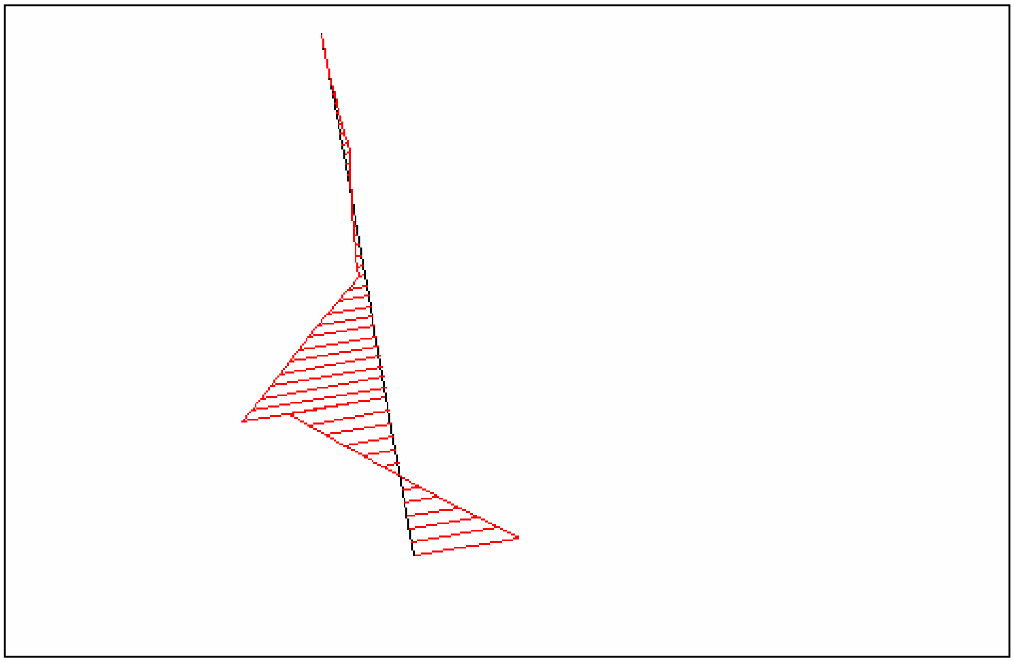


Fig. 7.14 Whole Structure Mast (Bending Moment)

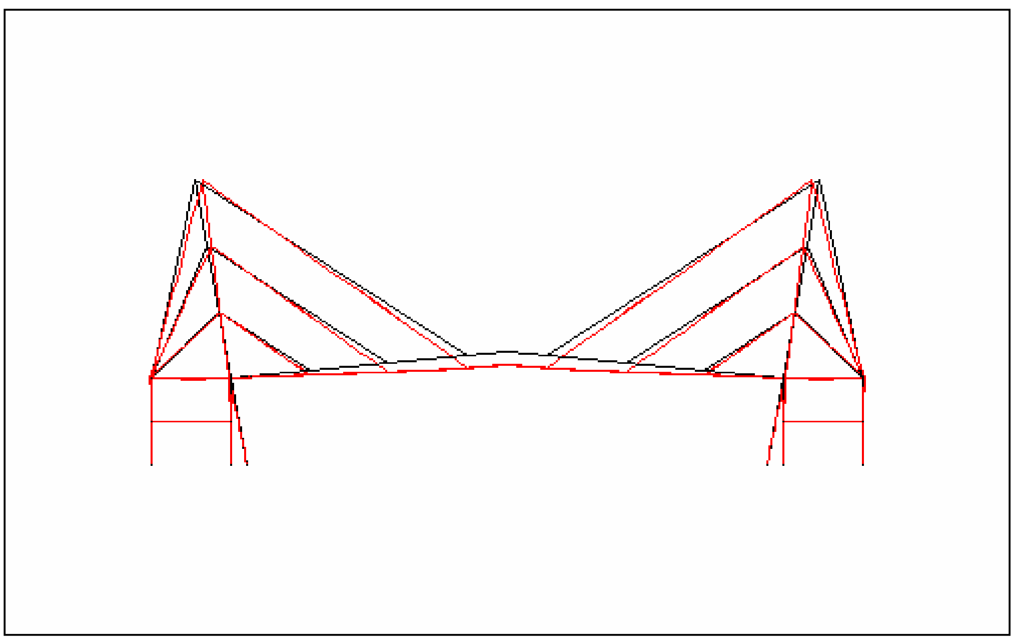


Fig. 7.15 Deflected Shape Whole Structure

Table 7.8 Mode Shapes

MODE	FREQUENCY (CYCLE / SEC)	PERIOD (SEC)
1	0.534	1.87299
2	0.724	1.38076
3	1.306	0.76576
4	1.645	0.60777
5	1.816	0.55060
6	1.956	0.51116

MODE SHAPE 1

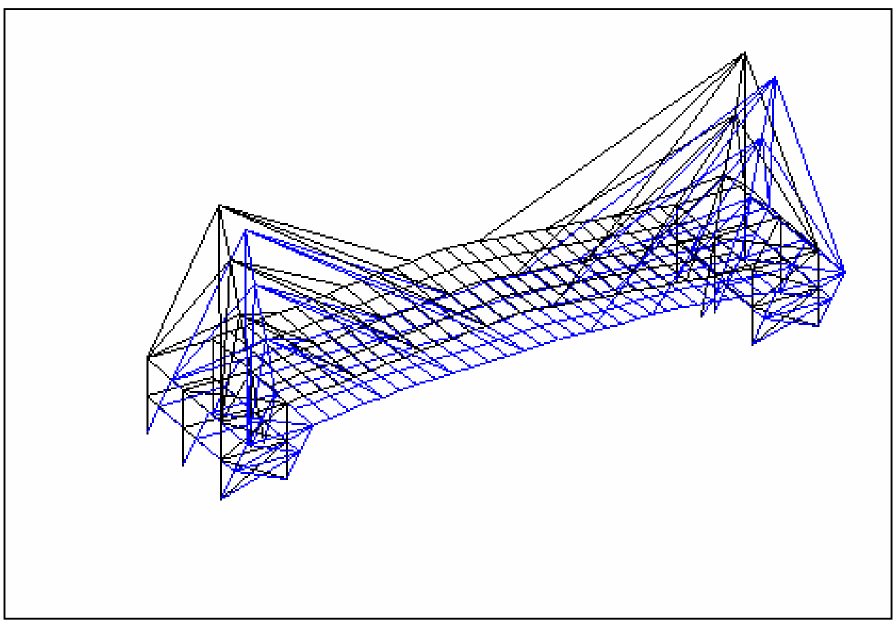


Fig. 7.16 Mode Shape 1 (Symmetrical Bending)

MODE SHAPE 2

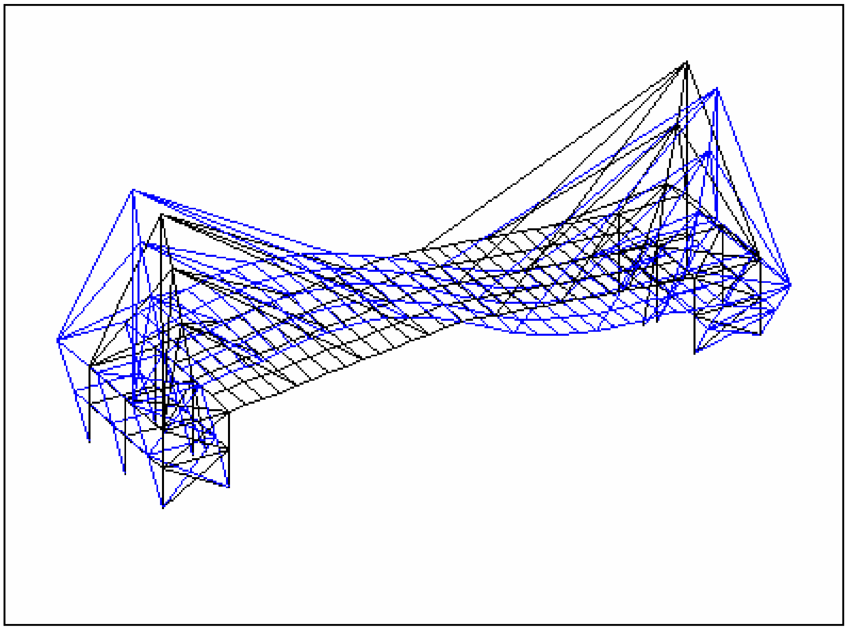


Fig. 7.17 Mode Shape 2 (Asymmetrical Bending)

MODE SHAPE 3

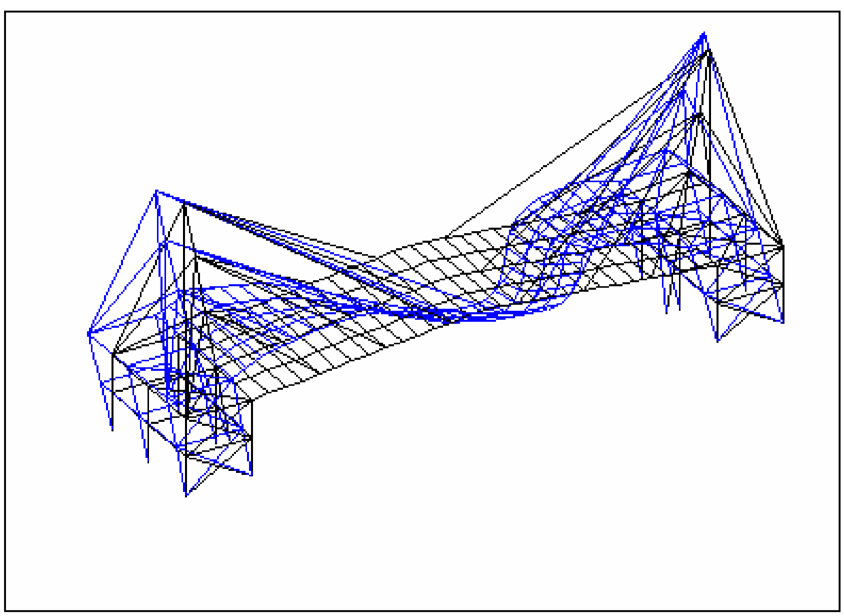


Fig. 7.18 Mode Shape 3 (Torsion)

CHAPTER 8 STATIC NONLINEAR ANALYSIS RESULT

The analysis results obtained after non-linear static analysis with different load cases using STAAD.Pro are listed in tabulated form in the following pages.

The results of Maximum Cable Force Fx in kN, Maximum Mast Force in kN, Maximum Mast Moment in kN-m, Maximum Main Girder Force in kN, Maximum Main Girder Moment in kN-m, Maximum Purlin Force in kN and Maximum Purlin Moment in kN-m, Maximum Tie-back Frame Force in kN and Max Tie-back Frmae moment in kN-m are sorted from the analysis results obtained from the STAAD.Pro.

From studying analysis results we can see that for most of all component parts of the structure the dead load + live load + wind load in x direction is the ruling force combination and for that basic design is prepared.

Static non-linear analysis results are discussed for the following components.

- Cable Force
- Main Girder Force
- Main Girder Moment
- Mast Force
- Mast Moment
- Purlin Transverse Beam Force
- Purlin Moment
- Tie-back Column Force
- Tie-back Column Moment

Table 8.1 Results of Static Non-Linear Analysis for Cables

	Load Case 9	Load Case 10	Load Case 14	Load Case 29
Max Tension (Fx) kN	309.76	376.76	361.08	311.22

Table 8.2 Results of Static Non-Linear Analysis for Main Girder

	Load Case 9	Load Case 10	Load Case 14	Load Case 29
Max Axial Force (Fx) kN	411.46	521.57	449.75	414.52
Max Bending Moment (Mz) kN-m	187.78	225.08	196.81	186.78

Table 8.3 Results of Static Non-Linear Analysis for Purlin

	Load Case 9	Load Case 10	Load Case 14	Load Case 29
Max Axial Force (Fx) kN	5.49	6.81	5.04	5.49
Max Bending Moment (Mz) kN-m	12.38	16.30	14.58	12.33

Table 8.4 Results of Static Non-Linear Analysis for Mast

	Load Case 9	Load Case 10	Load Case 14	Load Case 29
Max Axial Force (Fx) kN	2080.08	2490.16	2318.84	2084.87
Max Bending Moment (Mz) kN-m	104.81	135.32	134.09	105.41

Table 8.5 Results of Static Non-Linear Analysis for Tie-Back Column

	Load Case 9	Load Case 10	Load Case 14	Load Case 29
Max Axial Force (Fx) kN	592.53	714.98	656.82	596.86
Max Bending Moment (Mz) kN-m	82.27	103.10	91.79	79.92

Note:

Load case 9 Dead Load + Live Load
Load case 10 Dead Load + Live Load + Wind Load (pressure) in X Direction
Load case 14 Dead Load + Live Load + Wind Load (Half Pressure and Suction in X Direction
Load case 29 Dead Load + Live Load + Earthquake in X direction

Table 8.6 Deflection of Non-Linear Static Analysis

NODE	X (mm)	Y (mm)	Z (mm)
692	1.27	-206.35	-0.0
495	22.03	-7.89	-1.41
244	-10.97	-75.44	0.40
496	77.15	-3.92	-1.96
245	-4.70	-144.37	0.18
497	129.14	2.53	-2.49
246	0.38	-197.45	0.07

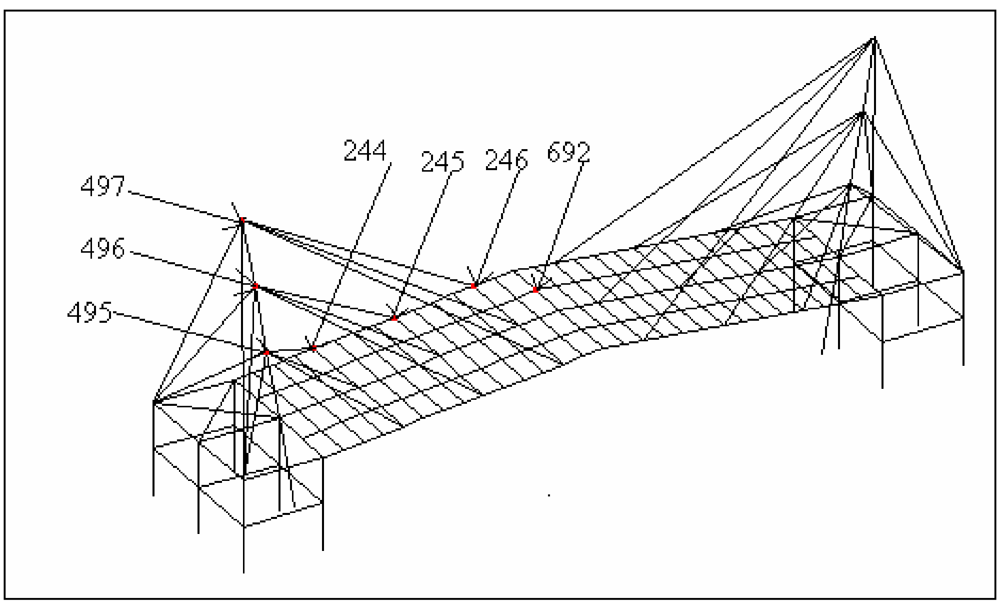


Fig. 8.1 Deflected Node Number

Table 8.7 Support Reactions

	Horizontal	Horizontal	Vertical	Horizontal	Moment		
Node	L/C	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
451	12	220.32	-352.74	-1.31	-2.75	-0.00	-38.69
23	10	-224.12	-361.41	1.03	1.60	-0.00	39.47
296	10	-19.96	1717.47	-0.42	-0.29	0.00	98.20
253	10	16.69	-703.93	0.02	-0.00	0.00	-81.79
493	10	-140.08	900.73	2.32	12.36	2.18	-88.89
486	12	134.90	877.79	-2.67	-14.21	2.51	94.66
493	10	-140.08	900.73	2.32	12.36	2.18	-88.89
486	12	134.90	877.79	-2.67	-14.21	2.51	94.66
486	12	134.90	877.79	-2.67	-14.21	2.51	94.66
493	37	-36.06	183.73	-0.72	-3.88	-0.68	20.85
294	10	-21.00	81.60	-0.12	-0.01	0.00	99.50
175	12	20.30	80.35	-0.19	-1.26	0.00	-95.57

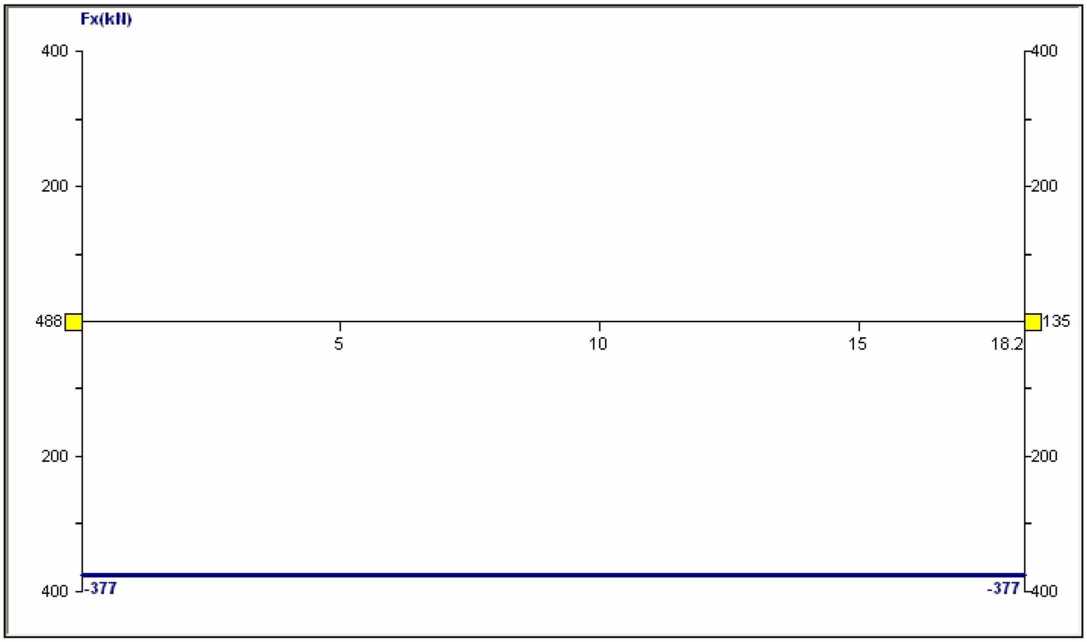


Fig. 8.2 Cable (Axial Force)

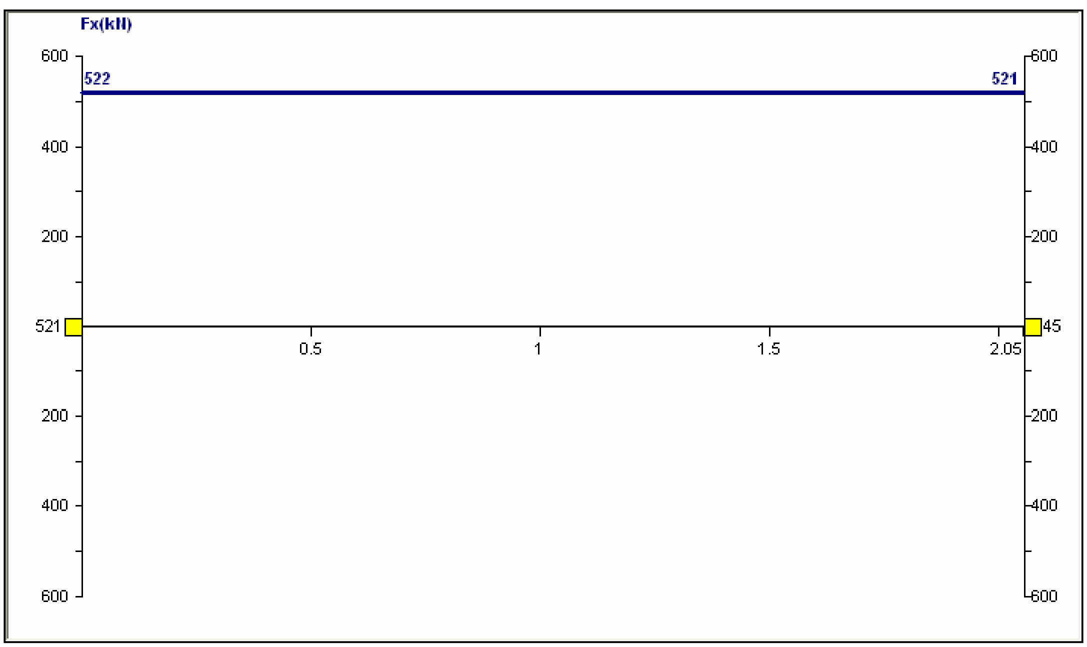


Fig. 8.3 Main Girder (Axial Force)

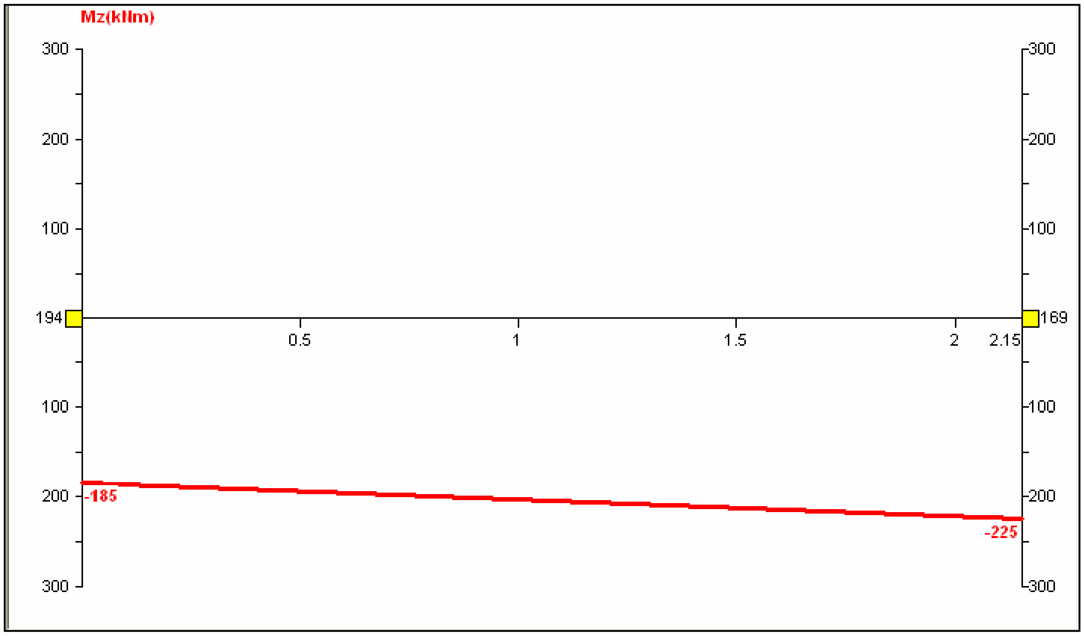


Fig. 8.4 Main Girder (Bending Moment)

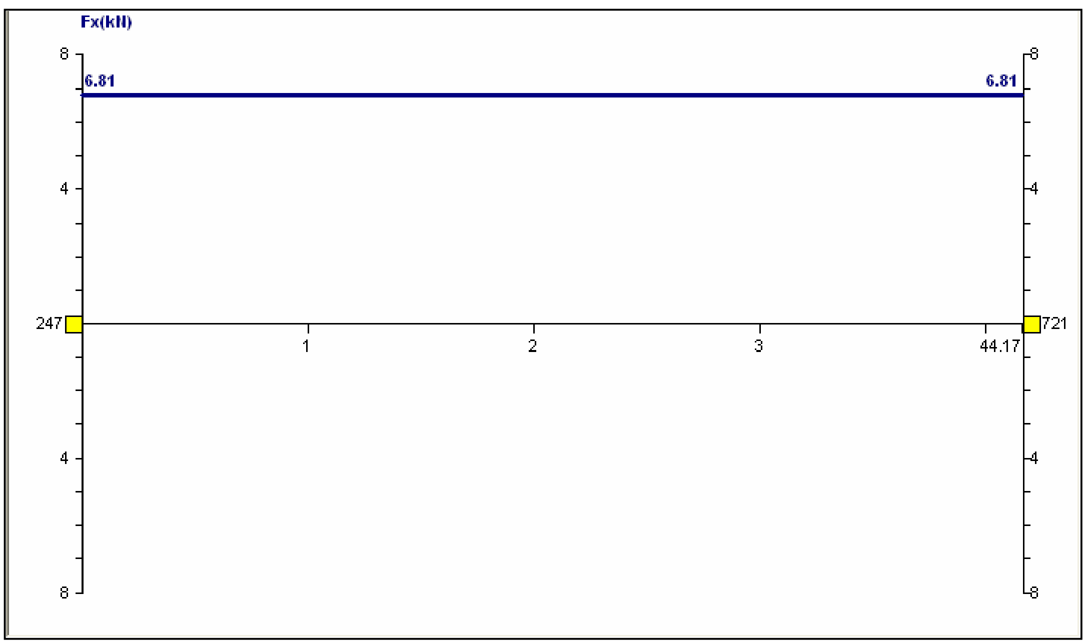


Fig. 8.5 Purlin (Axial Force)

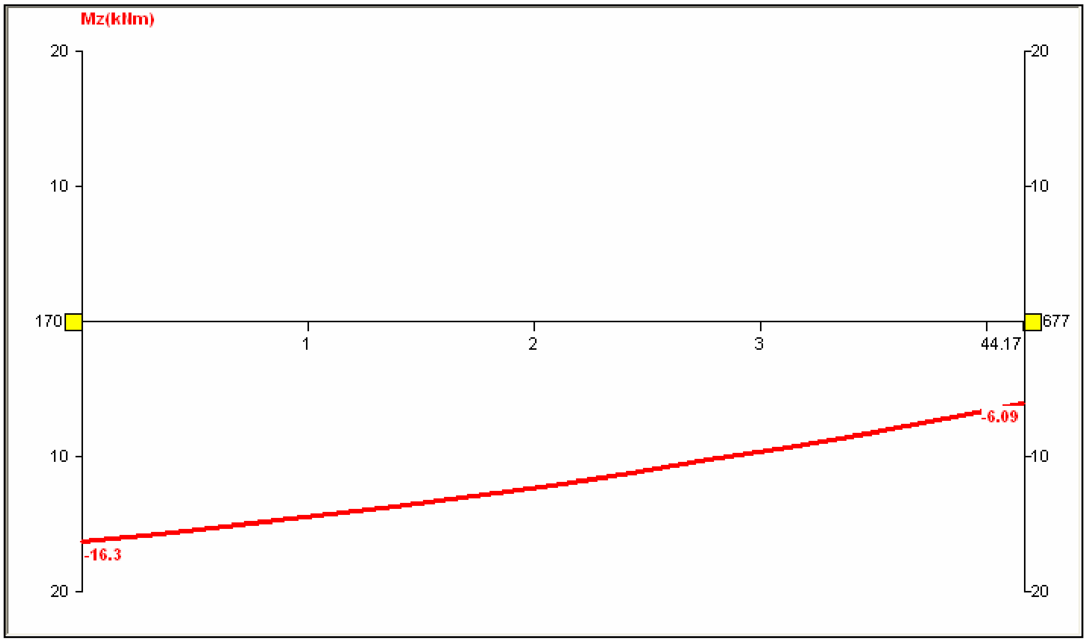


Fig. 8.6 Purlin (Bending Moment)

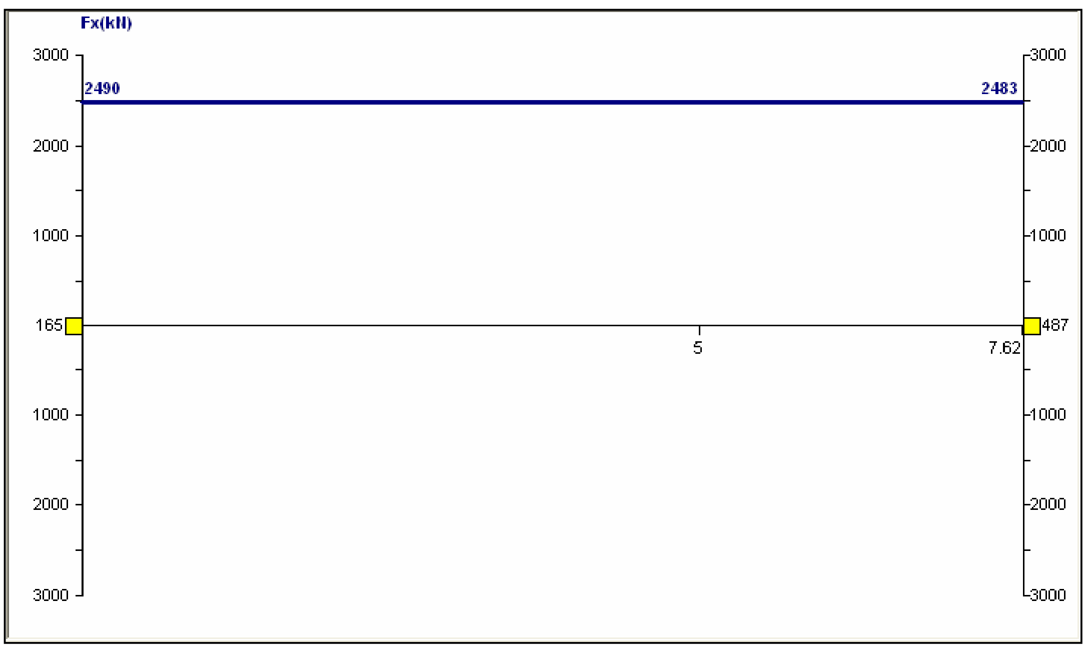


Fig. 8.7 Mast (Axial Force)

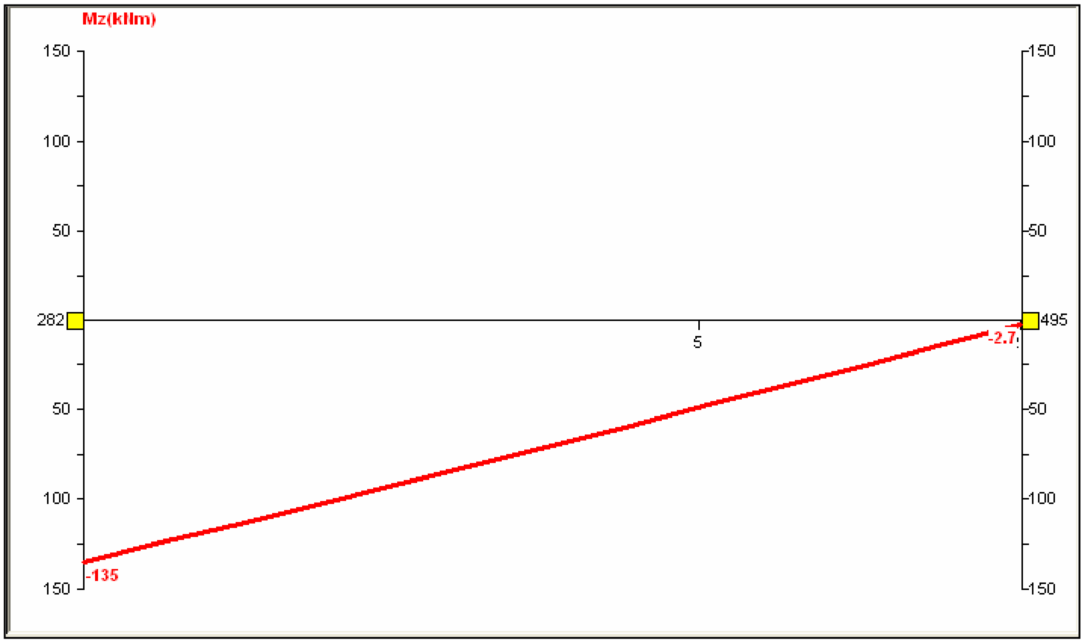


Fig. 8.8 Mast (Bending Moment)

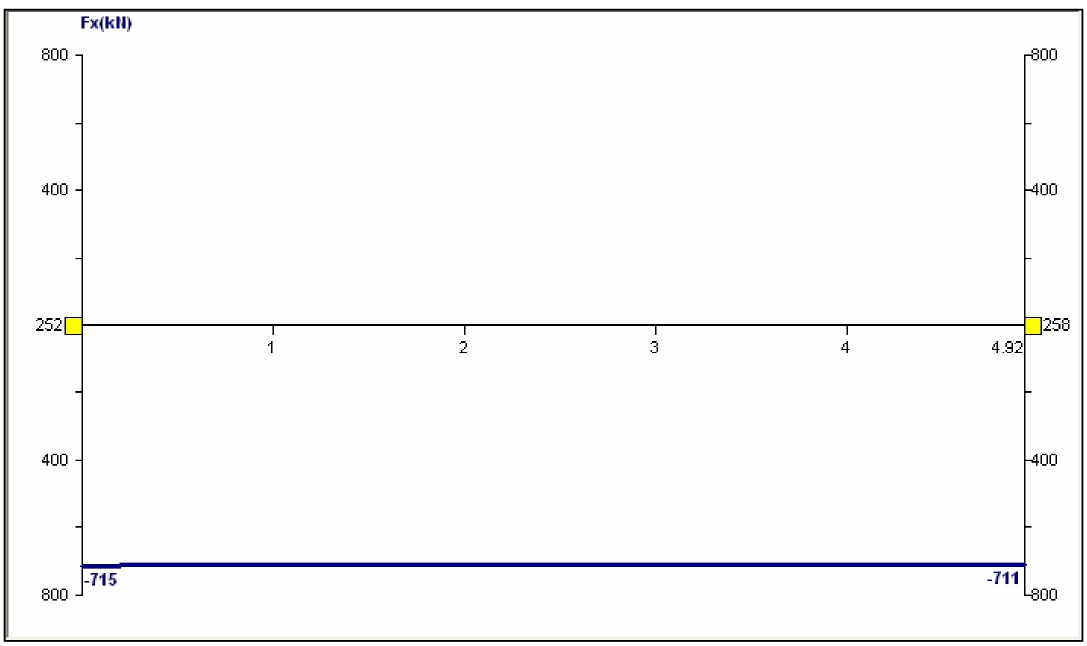


Fig. 8.9 Tie-Back Frame (Axial Force)

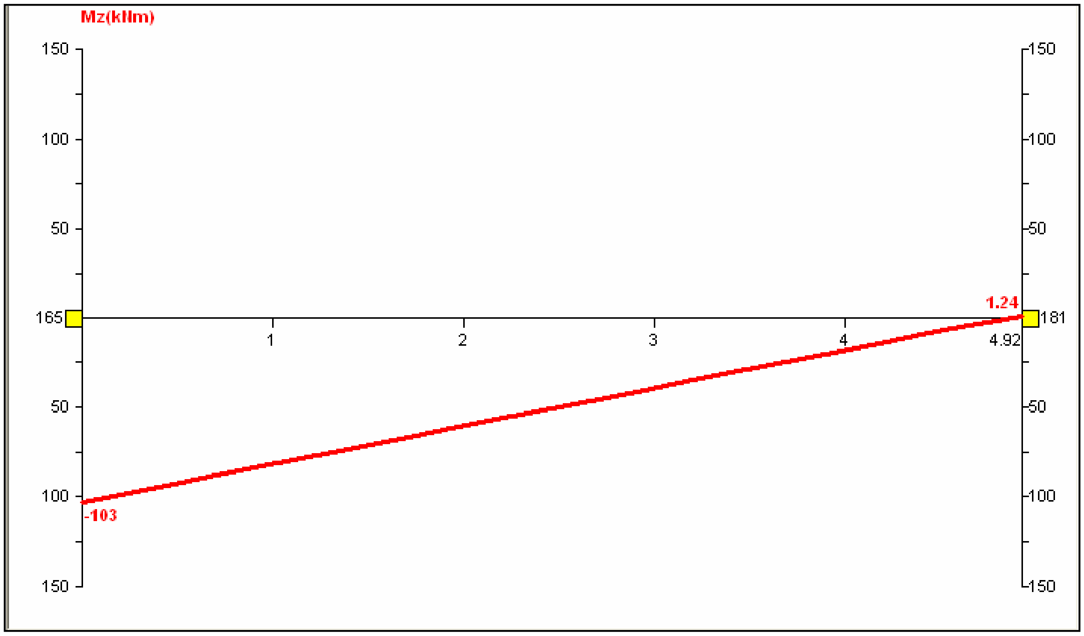


Fig. 8.10 Tie-Back Frame (Bending Moment)

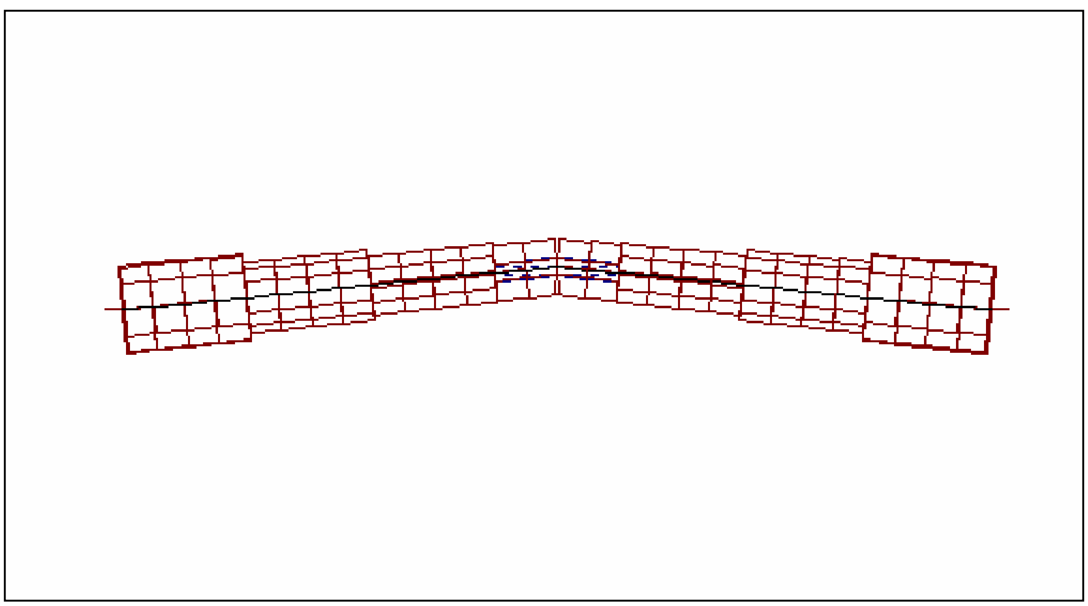


Fig. 8.11 Whole Structure Beam (Axial Force)

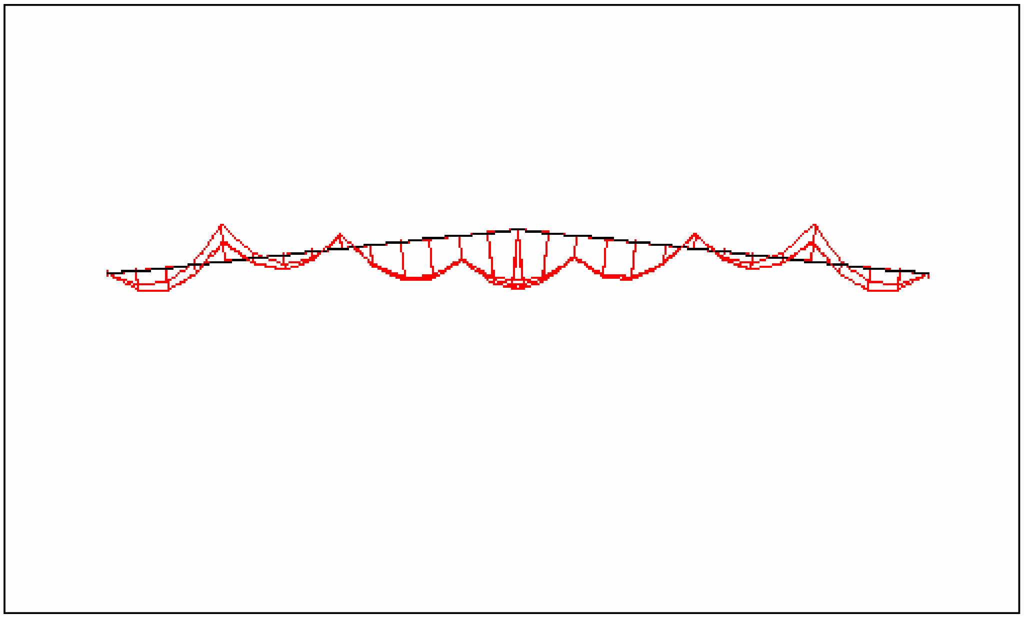


Fig. 8.12 Whole Structure Beam (Bending Moment)

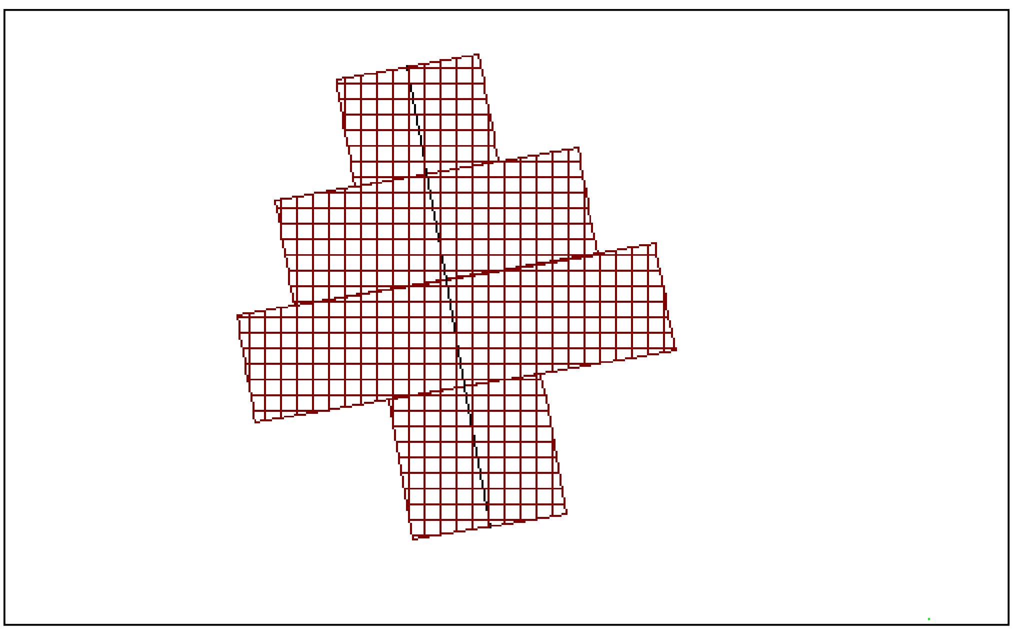


Fig. 8.13 Whole Structure Mast Axial Force

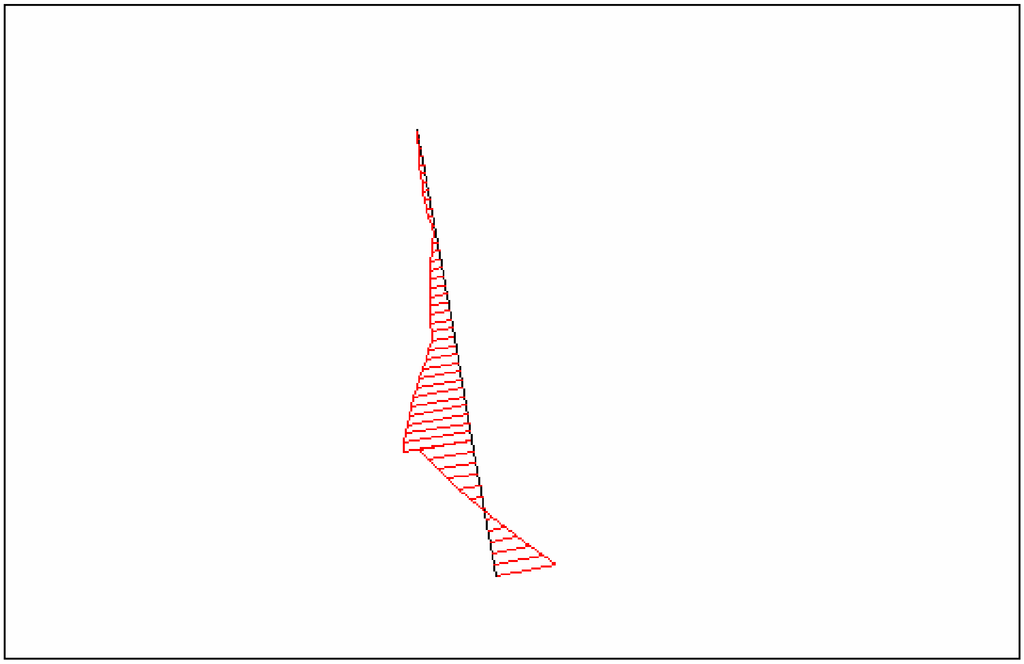


Fig. 8.14 Whole Structure Mast (Bending Moment)

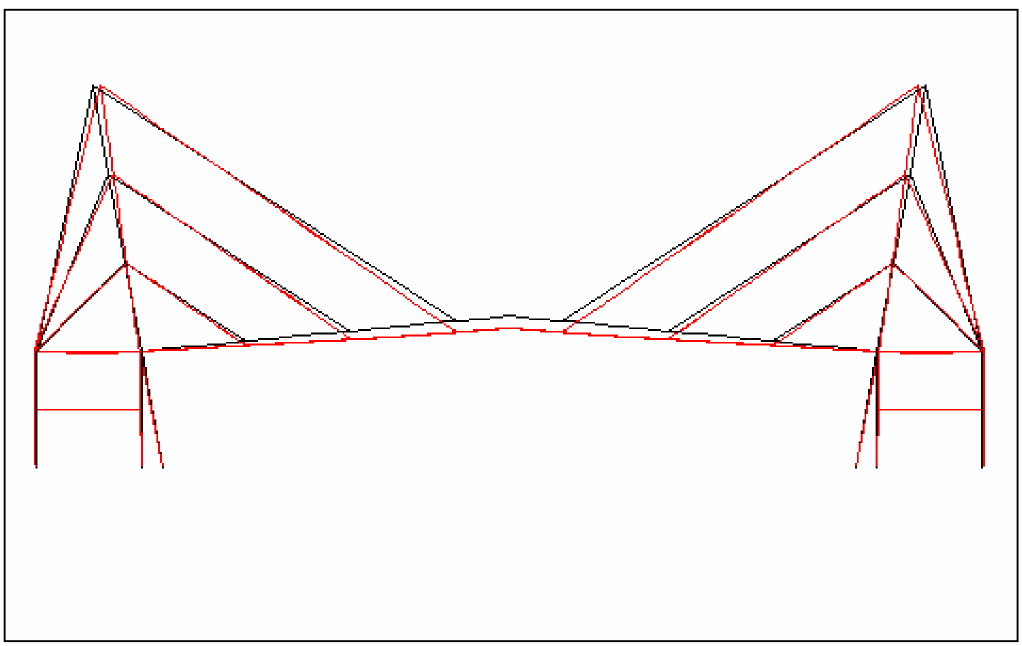


Fig. 8.15 Deflected Shape Whole Structure

Table 8.8 Comparison of Deflections of Linear and Non-Linear Static Analysis

Nodes	X (mm)			Y (mm)		
	Linear	Non-Linear	% Age Difference	Linear	Non-Linear	% Age Difference
692	1.52	1.27	16.44	-340.82	-206.35	39.45
495	29.19	22.03	24.53	-7.89	-7.65	3.05
244	-19.40	-10.97	43.45	-119.85	-75.44	37.05
496	114.49	77.15	32.61	0.60	-3.92	84.69
245	-8.55	-4.70	45.02	-236.54	-144.37	38.96
497	196.38	129.14	34.24	11.93	2.53	78.79
246	0.38	0.11	71.05	-326.69	-197.45	39.56

Table 8.9 Comparison of Forces of Linear and Non-Linear Static Analysis

Members	Maximum Force (kN)		
	Linear	**Non-Linear**	**% Age Difference**
Cable			
Max Tension (Fx) kN	476.51	376.76	20.93
Main Girder			
Max Axial Force (Fx) kN	765.00	521.57	31.82
Purlin			
Max Axial Force (Fx) kN	10.81	6.81	37.00
Mast			
Max Axial Force (Fx) kN	2894.59	2490.16	13.97
Tie-back Frame			
Max Tension (Fx) kN	874.84	714.98	18.27

Table 8.10 Comparison of Results with Different Load Steps for Cable and Mast

Load Step	Deflection	Force	
		Cable	Mast
50	34.25	113.47	1058.25
60	41.74	125.41	1142.22
70	50.32	139.63	1232.15
80	60.44	156.49	1332.79
90	72.53	176.62	1445.47
100	86.87	200.02	1572.54
110	104.72	228.33	1722.29
120	127.64	263.42	1904.81
130	158.62	309.12	2141.08
140	206.35	376.76	2489.92

9.1 DESIGN OF CABLE MEMBER

Maximum Axial Force in cable	=	376.76	kN
Ultimate Tensile Strength (UTS)	=	1520	N/mm^2
Total length of all cables	=	2691	m
Area required	=	$\frac{P}{UTS}$	
A	=	247.87	mm^2

9.2 DESIGN OF MAIN GIRDER

Intermediate Beam

load case 10

Axial force	=	521.57	kN
Shear Force	=	18.91	kN
moment about x-x	=	225.08	kN-m
moment about y-y	=	4.85	kN-m
condition for Cm			
in x dir.	=	0.85	
in y dir.	=	0.85	
		side sway is not prevented	

Assume steel section ISMB600

A	=	15621	mm^2
D	=	600	mm
B_f	=	210	mm
t_f	=	20.8	mm
t_w	=	12.3	mm
I_{xx}	=	9.18E+08	mm^4
I_{yy}	=	2.65E+07	mm^4
Z_{xx}	=	3.06E+06	mm^3
Z_{yy}	=	2.53E+05	mm^3

r_{xx} = 242.40 mm

r_{yy} = 41.20 mm

L_{effx} = 2150 mm

L_{effy} = 2150 mm

E = 2.00E+05 N/mm^2

fy = 2.50E+02 N/mm^2

$$\frac{T}{t_w} = \frac{20.8}{12.3} = 1.69 \quad < 2$$

$$\frac{d_1}{t_w} = \frac{558.4}{12.3} = 45.40 \quad < 85$$

$$\frac{D}{T} = \frac{600}{20.8} = 28.85$$

$$\frac{l}{r_{yy}} = \frac{2150}{41.20} = 52.18 \quad \text{since} < 180 \text{ o.k.}$$

$$\frac{l}{r_{xx}} = \frac{2150}{242.40} = 8.87 \quad \text{since} < 180 \text{ o.k.}$$

INETRMEDIATE FRAME

Check for bending stresses

f_{bcx} perm. = 154 N/mm^2 Refer IS 800 cl 6.1

$$f_{bcx} \text{ calc.} \quad \frac{Mxx}{Z_{xx}}$$

f_{bcx} calc. = 73.54 N/mm^2 since < perm. O.K.

f_{bcy} perm. = 165 N/mm^2

$$f_{bcy} \text{ calc.} \quad \frac{Myy}{Z_{yy}}$$

f_{bcy} calc. = 19.21 N/mm^2 since < perm. O.K.

Check for axial compression

f_{ac} perm. = 129.71 N/mm^2 Refer IS 800 cl5.1.1

f_{ac} cal. = 33.39 N/mm^2 since < perm. O.K.

$$f_{ac}\text{ perm.} = \frac{0.6\times f_{cc}\times f_y}{[(f_{cc})^{(n)}+(f_y)^n]^{(1/n)}}$$

f_{cc} = 724.11 $[\pi^2\times E/\lambda^2]$ N/mm^2

n = 1.40

f_{ac} perm. = 129.71 N/mm^2

Check for combined stresses

As per IS 800 - 1984 cl.7.1.1

$$\text{if } \frac{f_{ac}\text{cal}}{f_{ac}\text{perm}} \le 0.15$$

$$\frac{f_{ac}\text{cal}}{f_{ac}\text{perm}} + \frac{f_{bcx}\text{calc.}}{f_{bcx}\text{perm.}} + \frac{f_{bcy}\text{calc.}}{f_{bcy}\text{perm.}} \le 1$$

$$\text{if } \frac{f_{ac}\text{cal}}{f_{ac}\text{perm}} \ge 0.15$$

$$\frac{f_{ac}\text{cal}}{f_{ac}\text{perm}} + \frac{C_{mx}\times f_{bcx}\text{calc.}}{\left\{1-\frac{f_{ac}\text{cal}}{0.60f_{ccx}}\right\}f_{bcx}\text{perm.}} + \frac{C_{my}\times f_{bcy}\text{calc.}}{\left\{1-\frac{f_{ac}\text{cal}}{0.60f_{ccy}}\right\}f_{bcy}\text{perm.}} \le 1$$

C_{mx} = 0.85

C_{my} = 0.85

f_{ccx} = 25091.00 N/mm^2

f_{ccy} = 724.85 N/mm^2

$$\frac{f_{ac}\text{ cal.}}{f_{ac}\text{ perm.}} = \frac{33.3890276}{129.711588} = 0.26 \quad > 0.15$$

$$\frac{f_{ac}cal}{f_{ac}perm} + \frac{C_{mx} \times f_{bcx}calc.}{\left\{1 - \frac{f_{ac}cal}{0.60f_{ccx}}\right\} f_{bcx}perm.} + \frac{C_{my} \times f_{bcy}calc.}{\left\{1 - \frac{f_{ac}cal}{0.60f_{ccy}}\right\} f_{bcy}perm.} \leq 1$$

= 0.26 + 0.5 + 0.2

= 0.96 since $\leq$ 1 Assumed section is ok

9.3 DESIGN OF MAST

Load case	10		
Axial force		2490	kN
moment about x-x		135.32	kN-m
moment about y-y		6.486	kN-m
condition for Cm			
	in x dir.	0.6	
	in y dir.	1	

Assume steel section 50030025

A	=	37500	mm^2
D	=	500	mm
B_f	=	300	mm
t_f	=	25	mm
t_w	=	25	mm
I_{xx}	=	1.23E+09	mm^4
I_{yy}	=	5.39E+08	mm^4
Z_{xx}	=	4.91E+06	mm^3
Z_{yy}	=	3.59E+06	mm^3
r_{xx}	=	180.86	mm
r_{yy}	=	119.90	mm
L_{effx}	=	10000	mm
L_{effy}	=	10000	mm
E	=	2.00E+05	N/mm^2
fy	=	2.50E+02	N/mm^2

$$\frac{T}{t_w} = \frac{25}{25} = 1.00 < 2$$

$$\frac{d_1}{t_w} = \frac{450}{25} = 18.00 \quad < 85$$

$$\frac{D}{T} = \frac{1}{25} = 0.04$$

$$\frac{l}{r_{yy}} = \frac{10000}{119.90} = 83.41 \quad \text{since} < 180 \text{ o.k.}$$

$$\frac{l}{r_{xx}} = \frac{10000}{180.86} = 55.29 \quad \text{since} < 180 \text{ o.k.}$$

Check for bending stresses

f_{bcx} perm. = 139 N/mm^2 Refer IS 800 cl6.1

$$f_{bcx} \text{ calc.} \quad \frac{Mxx}{Z_{xx}}$$

f_{bcx} calc. = 27.58 N/mm^2 since < permO.K.

f_{bcy} perm. = 165 N/mm^2

$$f_{bcy} \text{ calc.} = \frac{Myy}{Z_{yy}}$$

f_{bcy} calc. = 0.00 N/mm^2 since < permO.K.

Check for axial compression

f_{ac} perm. = 97.08 N/mm^2 Refer IS 800 cl5.1.1

f_{ac} cal. = 66.40 N/mm^2 since < per O.K.

$$f_{ac} \text{ perm.} = \frac{0.6 \times f_{cc} f_y}{[(f_{cc})^{(n)} + (f_y)^n]^{(1/n)}}$$

f_{cc} = 283.46 $[\pi^2 \times E/\lambda^2]$

n = 1.40

f_{ac} perm. = 97.08 N/mm^2

Check for combined stresses

As per IS 800 - 1984 cl.7.1.1

if $\dfrac{f_{ac}cal}{f_{ac}perm} \leq 0.15$

$$\frac{f_{ac}cal}{f_{ac}perm} + \frac{f_{bcx}calc.}{f_{bcx}perm.} + \frac{f_{bcy}calc.}{f_{bcy}perm.} \leq 1$$

if $\dfrac{f_{ac}cal}{f_{ac}perm} \geq 0.15$

$$\frac{f_{ac}cal}{f_{ac}perm} + \frac{C_{mx} \times f_{bcx}calc.}{\left\{1 - \dfrac{f_{ac}cal}{0.60f_{ccx}}\right\}f_{bcx}perm.} + \frac{C_{my} \times f_{bcy}calc.}{\left\{1 - \dfrac{f_{ac}cal}{0.60f_{ccy}}\right\}f_{bcy}perm.} \leq 1$$

C_{mx} = 1

C_{my} = 0.6

f_{ccx} = 645.66 N/mm^2

f_{ccy} = 283.75 N/mm^2

$$\frac{f_{ac}\text{ cal.}}{f_{ac}\text{ perm.}} = \frac{66.40}{97.08} = 0.68 > 0.15$$

$$\frac{f_{ac}cal}{f_{ac}perm} + \frac{C_{mx} \times f_{bcx}calc.}{\left\{1 - \dfrac{f_{ac}cal}{0.60f_{ccx}}\right\}f_{bcx}perm.} + \frac{C_{my} \times f_{bcy}calc.}{\left\{1 - \dfrac{f_{ac}cal}{0.60f_{ccy}}\right\}f_{bcy}perm.} \leq 1$$

= 0.68 + 0.24 + 0

= 0.92 since $\leq$ 1 Assumed section is ok

CHAPTER 10 CONCLUSION AND FURTHER SCOPE OF WORK

10.1 SUMMARY

In the present study the geometry of cable-supported roof has been developed using STAAD.Pro 2003 stiffness based software. STAAD.Pro 2003, a general stiffness based program has been used to solve the static linear and static non-linear analysis problem for the cable-supported roof.

For the analysis of the cable supported roof, span of the roof is taken as 60m. Width of roof is taken 100m. Height of mast is 32.86m at 10 degree inclination. Girders are spaced at 8.33m centre to centre while supporting frame or tie-back frame is at 8.61m either side of the mast. Purlin is spaced at every 2.15m centre to centre of girder. Cables anchored outward from the mast and support roof girders located at 8.61m to either side of the central girder. 9 forestay and 6 backstay cables are anchored outwards at 7.62m height of mast at three levels.

Basically a cable supported roof is a long span and light weight structure. In the present study weight of sheeting is taken as 0.2 kN/m^2, while live load is taken as 0.75 kN/m^2. Instead of uniform distributed loads, the entire load applied as a joint load. Dynamic load such as Wind load and earthquake load is also considered for the analysis. Calculation of wind load is done by the gust effectiveness factor method. At roof, wind load is applied with pressure and suction coefficient. While at mast wind load is acting as a nodal load. Seismic forces are calculated by the response spectrum method. Different load combinations have been also applied to the structure for finding out critical load combination.

After the analyzed the structure in STAAD.Pro 2003, results of static linear and static non-linear of cable, main girder, purlin, mast and tie-back frame are compared for dead load + live load, dead load + live load + wind load (pressure) in x direction, dead load + live load + wind load (half pressure and half suction) in x direction and dead load + live load + earthquake in x direction. Results of static linear and static non-linear of nodal deflection are also compared.

10.2 CONCLUSION

The following conclusions are made on the work done.

- On comparing the results obtained after performing a linear static analysis and non-linear static analysis, it is conclude that cable-supported roof is a highly non-linear structure. The forces obtained in the members when non-linear analysis is done, are lesser than that obtained from the linear analysis. Hence, when the members are designed for the forces from non-linear analysis, are of lighter section and the overall weight of the structure is reduced.

- The convergence criteria and the load step highly beneficial to get a converged solution. It is useful to reduce the amount of initial load in to the structure until it has converged, then to progressively increase the load, instead of applying it all at once.

- Deflections are lesser in the static non-linear analysis compared to static linear analysis. Nonlinearity is more effective in the roofs where cables are not pretensioned.

- Cable-supported systems can be used to create dramatic and inspirational structures that enclose large volume column-free spaces, and still provide unique opportunities for architectural design freedom.

- Cable supported roof is a versatile structure can be used to replace existing roof without hindering working of the structure.

- The backstay cables transfer load to steel tie-back column, is produce negative axial force or tension in the column which in turn anchor to massive concrete foundations resisting the overturning forces from the weight of the roof.

- To avoid the suction effect of wind load in cable-supported roof dead weight of roof is increased or may also anchored suspender at appropriate girder location.

10.3 FURTHER SCOPE OF WORK

Various topics, on which, further work can be carried out, are summarized as follows:

- Parametric study with different span to height ratio and with different cable arrangement can also be done.
- Dynamic analysis can be done in cable-supported roof.
- Aerodynamic behaviour of cable-supported roof and hence fluttering effect is also considered for the design purpose.
- Secondary loads like temperature and creep effect is also considered for the design purpose.
- Design and detailing of the component parts of the cable supported roof and comparison of cost with the conventional roof structure.

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