ANALYSIS AND DESIGN OF CABLE STAYED BRIDGE

By

Blesson Thomas.B

08MCL017

DEPARTMENT OF CIVIL ENGINEERING

AHMEDABAD-382481

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Major Project

Submitted in partial fulfillment of the requirements

For the degree of

Master of Technology in Civil Engineering

(Computer Aided Structural Analysis & Design)

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Declaration

This is to certify that

i) The thesis comprises my original work towards the degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) at Nirma University and has not been submitted elsewhere for a degree.

ii) Due acknowledgement has been made in the text to all other material used.

Blesson Thomas.B

Certificate

This is to certify that the Major Project entitled "Analysis and Design of Cable Stayed Bridge" submitted by Blesson Thomas (08MCL017), towards the partial fulfillment of the requirements for the degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) of Nirma University, Ahmedabad is the record of work carried out by him under my supervision and guidance. In my opinion, the submitted work has reached a level required for being accepted for examination. The results embodied in this major project, to the best of my knowledge, haven't been submitted to any other university or institution for award of any degree or diploma.

Prof. S. P. Thakkar
Guide,
Asst. Professor,
Department of Civil Engineering,
Institute of Technology,
Nirma University, Ahmedabad

Dr. P. H. Shah
Professor and Head,
Department of Civil Engineering,
Institute of Technology,
Nirma University,
Ahmedabad

Dr K Kotecha
Director,
Institute of Technology,
Nirma University,
Ahmedabad

———————
Examiner

———————
Date of Examination

Abstract

Man has achieved a lot in field of Structural Engineering as it is most evident in the World's largest bridge spans, tallest structures etc. In the recent years cable stayed bridges have received more attention than any other bridge mainly, in the United States, Japan and Europe as well as in third-world counties due to their ability to cover large spans. Cable-stayed can cross almost 1000m (Tatara Bridge, Japan, Normandie Bridge, France)

In India few of the cable stayed bridges are constructed and a couple of them are underway. Like Bandra-Worli sea link, Second Hoogly Bridge are the finest example of application of cable stayed bridge in India. Cable stayed bridges for road over bridge in Bangalore and Chennai have come up and a cable stayed road over bridge is proposed in various smaller emerging cities.There is still place for innovation in Cable-stayed bridge techniques

Here detail study of the cable stayed bridge is done. Historical facts, various components of the bridges, types of the pylon arrangements, type of the cable configuration arrangements has been done to give a brief idea about cable stayed bridge from the various literature surveyed.

For validation of the software used an example solved in a technical paper has taken and verified.Cable stayed bridge of 200m span having 100m central span with H type pylon shape and with semi-harp type cable configuration has been taken for analysis in SAP2000.

The cable stayed bridge is modeled with proper technique from the guidance of various literature surveyed. Then study for the behavior under various loading is undertaken. Live load was taken according to IRC 6:2000. IRC Class A and Class 70R vehicle load along with Aerostatic wind loads was undertaken.

A Dynamic analysis in the form of linear Time-History is also carried out to investigate dynamic response of the Cable-stayed Bridges and various response quantities such as Bending-moment, Shear-force,Torsion and Axial force are represented.

Then a Parametric Study has been carried out by varying the shape of the pylon and the central span of the bridge.The shape here taken for study are H shape, A shape, Diamond shape, Delta shape, Inverted Y shape. The span taken for study varied from 200m,400m,600m,800m. The linear and dynamic case has been compared for the response quantities.The result of study is represented in graphical form for various response quantities like Bending-moment, Shear-force, Torsion and Axial force.The performance of different shape are better for different response quantities so it cannot be merely concluded form analysis which is better so a detailed cost comparison can only give the better shape suitable by designing each shape and components.

Design of the cable stayed bridge is performed for 200m H type Cable stayed bridge. The cable are designed by taking maximum axial force acting on it and the providing the number of cables required for resisting the force. Pylon are designed as slender column with biaxial moment and axial force. Box Girder is designed as Pre-Tensioned three cell Box Girder. Pile foundation is provided as foundation.

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List of Nomenclature

An .... Vertical projected area of bridge.

A .... Cross sectional area.

A_g .... Gross cross section area of steel.

$A_s v$ .... Area of Shear reinforcement.

B .... Width.

$C_y(\alpha)$ .... Coefficient of drag force.

$C_z(\alpha)$ .... Coefficient of lift force.

$C_m(\alpha)$ .... Coefficient of pitching moment.

C_D .... Static aerodynamic coefficient in wind axis.

C_L .... Static aerodynamic coefficient in wind axis.

C_M .... Static aerodynamic coefficient in wind axis.

D .... Depth.

d_t .... Depth from extreme fibre.

e_0 .... Void ratio.

e_x .... Eccentricity in X-Direction.

e_y .... Eccentricity in Y-Direction.

E_s .... Modulus of elasticity of concrete.

E_s .... Modulus of elasticity of steel.

E_{eq} .... Equivalent cable modulus of elasticity.

f_{ck} .... Compressive strength of concrete.

f_{cj} .... Compressive strength of concrete at j day.

f_{cp} .... Compressive stress of centroidal axis due to prestressing force

f_t .... Max principle tensile stress in concrete.

f_y .... Characteristic strength of steel.

$F_y\ (\alpha)$ .... Drag force.

$F_z\ (\alpha)$ .... Lift force.

G .... Dead Load Bending Moment and Shear Force.

SG .... SIDL Bending Moment and Shear Force.

Q .... Live Load Bending Moment and Shear Force.

H .... Horizontal projected length of the cable.

k .... Wobble Coefficient.

k_1 .... Constant for slender column from SP 16.

k_2 .... Constant for slender column from SP 16.

K_1 .... Risk coefficient.

K_2 .... Terrain height and structure size factor.

K_3 .... Topography Factor.

L .... Span of bridge.

L_{ex} .... Effective Length in X-Direction.

L_{ey} .... Effective Length in X-Direction.

L_{grip} .... Grip length within Jack.

M(α) .... Pitching Moment.

M_f .... Max moment in pile.

M_t .... Cracking Moment.

M_g .... Moment due to Dead load.

M_q .... Moment due to live load.

M_u .... Factored Moment.

P .... Axial force in the cables.

P_{av} .... Average stress after frictional loss.

P_j .... Jacking Stress.

P_t% .... Percentage area of Steel.

P_u .... Factored Axial load on a compression member.

Q .... Discharge.

S .... Effective span.

t .... Temperature .

T .... Initial tensile force in the cable.

V_{co} .... Ultimate Shear Resistance.

V_z .... Design wind speed.
W_t .... weight of pile cap.
w .... Cable weight per unit length.
Z_b .... Section modulus at bottom.
Z_t .... Section modulus at bottom.
y_b .... C.G from bottom.
y_t .... C.G from top.
α .... Effective wind angle of attack.
α_t .... coefficient of Thermal Expansion.
α_0 .... Angle of inclination of wind.
θ .... Angle between the cables and bridge deck.
ρ .... Density of air kg/m^3.
μ .... Coefficient of Friction.
$\Delta\theta$ .... Cumulative angles.
σ_t .... Flexural stress at top of girder.
σ_b .... Flexural stress at bottom of girder.
δ .... Cable Elongation.
Δ .... Settlement of pile.
γ .... Density of soil.

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Chapter 1

Introduction

1.1 General

The achievement of man has been attributed to how large, long and tall he can create the structures around him. From the very beginning of the human race he has been trying to prove that he can create some very astonishing and amazing structures around him, like Pyramids of Egypt. The construction of cable stayed bridge of about 1000m span (Tatara Bridge, Japan) itself is an achievement in structural engineering.

After end of World War II there was shortage of construction materials like steel and cement thereby need to obtain optimum structural performance from these materials became imperative. New systems and technologies were evolved to meet these requirements. Cable stayed bridges are constructed along a structural system which comprises of a deck and continuous girders which are supported by stays in the form of cables attached to tower located at the main piers. Stiffness of the overall structure can be provided by stiff towers or can be stiffened by taking backstays to individual or by employing intermediate tension piers or combination of the stiffness of the main span, the tower and the back span.

Cable-stayed bridges,have become increasingly popular in the past decade in the United States, Japan and Europe as well as in third-world countries. This can be attributed to several advantages over suspension bridges, predominantly being associated with the relaxed foundation requirements, with the introduction of high-strength steel, development of welding technology and progress in structural analysis and new construction technique which is very much in vogue. The development and application of computers opened up new and practically unlimited possibilities for the exact solution of these highly statically indeterminate systems and for precise statically analysis of their three-dimensional performance. This leads to economical benefits which can favour cable-stayed bridges in free spans of up to 1000m.

1.2 Historical Review

The historical development occupies an important place in several publications dealing with cable-stayed bridges. So not mention it here will be a injustice to all those people who have been working hard for the development of structures like cable-stays bridges for betterment of society.

German engineer F. Dischinger rediscovered the stayed bridge, while designing a suspension bridge across the Elbe River near Humburg, Germany. He recognized that the inclined cables of the early cable stayed bridges were never subjected to axial tension, thus cables started to perform only after considerable deformation of the whole structure. In early development this type of behavior lead to the misconception that this type of bridges was unacceptable and so unsafe. During the World War II many bridge were destroyed in Germany. The demand to build these bridges was urgent. The requirements of efficient use of materials and speedy construction made cable stayed bridges the most economical design for the replacements.

The concept of cable stayed bridge, namely to support a beam stays from a pylon, can be dated back to 16th century. Some pedestrian cable stayed bridges were built during the period of 17th to 19th century. The cable stayed bridge initially was in bad press due to various failures of bridges like the one on river Tweed, England. The main reasons of such failure were the lack of adequate theoretical tools and techniques for the analysis of the structural behavior and lack of high strength materials.

The first 79m pedestrian Tweed River Bridge near Dryburgh-Abbev, England collapsed in 1818, for which no cause was reported and the second 78m Saale River Bridge in Nienburg,, Germany, collapsed in 1824 This collapse was reported to have been caused by an unusual crowd loading, which was not expected.

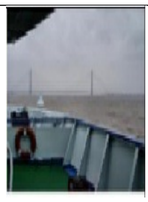
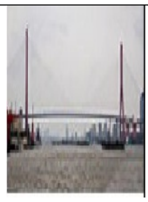
The state of art in the design and construction has changed immensely since the first steel cable-stay bridge was built in Stromsund, Sweden in 1955. This may be from the fact that the Stromsund Bridge had a span of 183m whereas Bridge of Japan which has been completed recently has a central span of 890m. Similarly the first modern prestressed concrete cable-stayed bridge was on Maracaibo Luke Bridge in Venezula completed 1962, and had five spans of 23.5m of each whereas the Chang Jiang second bridge of China is having a central span of 444m. Thus, the cable-stayed bridges are increasing not only in number and popularity worldwide but also in central span length.

The recent trend towards building longer cable-stay bridges raises new question of the validity of linear analysis to predict the response of the bridge structure. The increase in the cable sag can lead to a substantial decrease in the stiffness of the stay and subsequent nonlinear behavior of the bridge under live load has come into focus. The general trend amongst the designers is to use slender and streamlined bridge decks. With the lighter bridge deck, the live load becomes the larger percentage of the total load and hence fatigue has become an important factor.

The Port De-Normandie bridge in Le Harve, France was completed in 1995. The Suntong Bridge on Suzhou Nantong, in China has a longest sapn of about 1100m was completed in the year of 2008. While the Tatara Bridge at Seto island sea in Japan is about 900m and remained the longest cable stayed bridge for many years. Bridge of such long span length used to be the domain of the suspension bridge system. Modern cable stayed bridge has come a long way from its modest beginning in the form of Stromsund Bridge in Sweden in 1956. Today it has captured the imagination of mankind with large span and unimaginable shapes and forms. Such a revolution has happened due to the introduction of high strength cable and better construction techniques and analyzing tools readily available. The below Table1.1 show the top ten longest cable stayed bridges by its central span length.

Table 1.1: Cable stayed Bridges

Rank	Photo	Name	Country	Span	Year
1		**Sutong**	China	1088 m	2008
2		**Stonecutters**	Hong Kong	1018 m	2009
3		**Tatara**	Japan	890 m	1999
4		**Pont de Normandie**	France	856 m	1995
5		**3rd Nanjing Yangtze**	China	648 m	2005

Rank	Photo	Name	Country	Span	Year
6		**2nd Nanjing Yangtze**	China	628 m	2001
7		**Baishazhou**	China	618 m	2000
8		**Qingzhou Bridge**	China	605 m	2001
9		**Yangpu**	China	602 m	1993
10		**Bandra-Worli Sea Link**	India	600 m	2009

1.3 Cable Stayed Behavior

The basic concept is the utilization of high strength cables to provide intermediate supports for the girder so that it may span a much longer distance. The cable-stays are directly connected to the bridge deck resulting in much stiffer structure. A large number of closely spaced cable stays support the bridge deck through its length thus

reducing the required and bending stiffness of the longitudinal to the minimum and thus allowing easy construction of longer spans. The structural system is a triangulated force system in which cables are in tension while there is compression in pylons and deck. The secondary effects like bending, shear, deformation of structure plays pivotal role in stress determination of members and change the stresses considerably. However, Cable-stayed bridges can also be economically designed for medium and short spans by allotting appropriate stiffness to the cables, tower(s) and girder. Cable stayed bridges have therefore also been used for short span pedestrian bridges and aqueducts because of their pleasing form and are very quick to construct. There is still place for innovation in Cable-stayed bridge techniques, and the increase in their span length occurring during this last decade of the twentieth century is remarkable. The Cable-stayed bridge seems to be a developing bridge type of this era.
In early designs of modern cable-stayed bridges essentially consisted of a stiff girder support by a few cable. The stay forces were rather large and consequently the anchorage design was exclusively complex. Further development indicated that these problems could be eliminated by increasing the number of stays. The multi cable arrangement has following advantages:

a. The deck can be erected using a cantilever erection sequence in conjunction with suspension by successive cable-stays.

b. The use of a large number of small cables reduces the concentrated forces at the anchorage points in the tower and deck. Moreover the deck bending moments between suspension points are reduced.

c. A damaged or corroded cable-stay can be easily replaced without overstressing the bridge structure.

d. Excellent aerodynamic stability is obtained as the damping of the system in increased by adding a large of cables of different lengths with different natural frequencies.

For a multi-cable system, it is not necessary to have a stiff deck girder to carry the loads through a bending action but must have safety against buckling due to large longitudinal forces generated by the inclined stays and to limit local deformations under concentrated live loads.

1.4 Objective of Study

a. To study basic ideas and factors pertaining to cable stayed bridge.

b. To study the dynamic behavior of cable stayed bridge under seismic and wind effect.

c. To study the behavior of the cable stayed bridges for different shapes of pylons.

d. To study the length of central span affecting the cable stayed bridges.

e. Design of various components of cable stayeed bridge.

1.5 Scope of Work

a. Basic introduction, classification, loadings, advantages.

b. Study of various related topics of published papers from journals, magazines and books.

c. Analysis and design of a cable stayed bridge using SAP2000.

d. Parametric study.

- Comparison between various shapes like H-type, A-type, Inverted Y type, Delta type, and diamond shape type of pylons used
- Varying the Span of the bridge from 200m, 400m, 600m,800m

- Studying the effect on the shape of the pylons to the varying length of the span.

e. Aerostatic stability of cable stayed bridge.

f. Dynamic stability of cable stayed bridge.

1.6 Organization of Major Project

The rest of the thesis is organized as follows.
Chapter 1 gives information on the cable stayed bridges, various terminology, Historical reviews are done.
In **Chapter 2** literature survey is carried out on various topic related to cable stayed bridge. Various effects like the non-linearity, seismic effect, Live load conditions and some Experimental works has been done.
Chapter 3 gives detail about different types of cable stayed bridges has been compiled. The various classification based on the shape of pylons, arrangement of cable configuration are done and a verification example is done.
In this **Chapter 4** the method used for modeling a has been explained SAP2000. Then a cable stayed bridge with give problem data has taken for analysis by considering live loads, vehicular class loading according to IRC 6:2000 have been explained.
Chapter 5 studies about the various loading for. Aerostatic effects,time history analysis, combination has been done on a cable stayed bridge of 200m span and results are discussed.
In **Chapter 6**,study has been carried out by varying the shape of the pylon and the central span of the bridge.The shape here taken for study are H shape, A shape, Diamond shape, Delta shape, Inverted Y shape. The span taken for study varied from 200m,400m,600m,800m. The Linear and Dynamic case has been compared for the response quantities and are represented in graphical form.
Chapter 7 has been devoted for the designing aspect of cable stayed bridge.The

H-type pylon of 200m span has been considered for designing.Components like Cable, Pylon, Box Girder and Pile foundation has been done with their detailed drawings. In **Chapter 8** conclusion, summary and future scope of work has been discussed

Chapter 2

Literature Review

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 analysis, design of different cable stayed bridges with different configuration and support conditions. Several research papers and books have been studied for different methods of analysis of bridges superstructure.

2.2 Literature Review

2.2.1 Geometrical Nonlinearity and Aerodynamic on Cable Stayed Bridge

N. D. Shah And Dr. J. A. Desai [1] states the importance of nonlinearity on cable stayed bridges is emphasized.Span of the cable-stayed bridge increases, the nonlinearities also go on increasing which is due to sag in the cable, axial force-bending moment interaction in the girder and tower and due to large deformations of the overall structure. Further, the nonlinearity magnifies with the influence of wind loading. With the increase in bridge span, the diameter of the cable, the non linear wind structure interaction and the wind speed spatial non-uniformity increases, which have significant influence on the aerostatic behavior of long span cable-stayed bridges. This paper presents finite element approach for the geometric nonlinear aerostatic analysis of cable-stayed bridges with vehicular interaction and the concept of longer span is elaborated here with help of parametric study. The effect of anchoring top cables of cable stayed bride i.e. bi-stayed concept is also compared. The authors concludes that the concept of spread pylon proved useful in reducing the cable tensile forces whereas the bi-stayed bridge concept is useful in reducing the forces in cable, girder and pylons.

C. E. N. Mazzilli Et Al. [**2**] in their paper on studies aerodynamic forces for the classical flutter analysis. Reduction technique is applied to supply a set of two coupled homogenous equation of motion about the equilibrium configuration. The aerodynamic load is modeled within the finite element and is assumed to be in conformity with classical flutter analysis. The procedure is programmed using symbolic computation. For classical flutter case the system is modeled by at least two degree of freedom to account for typical model coupling. The aerodynamic model is assumed to have constant values for beam-element length l, the deck cross section with 2 C and wind velocity V along the span. The aerodynamic "stiffness" and "damping" nodal matrix are defined in terms of the vibration. The aerodynamic coefficient is determined in wind tunnel test. The presented method is applied to the determination of the critical wind velocity. Through the example the author concludes that with equivalent damping ratio ζ(V) increase with V, the circular frequency remains constant. In the examples solved the critical condition is not reached so the flutter phenomenon is not predicted in the selected wind velocity range this is because of the favorable geometrical characteristics of the bridge and short length of the deck and its cross section shape. The author further states that the same methodology to be applied to other structures which are more prone to flutter.

Pao-Hsii Wang et al [**3**] has discussed in their paper has made a comparison on nonlinear analysis of a high redundant cable-stayed bridge. The initial shapes including geometry and prestress distribution of the bridge are determined by using a two-loop iteration method. For the initial shape analysis a linear and a nonlinear computation procedure are set up. In the former all nonlinearities of cable-stayed bridges are disregarded, and the shape iteration is carried out without considering equilibrium. In the latter part the authors considered all nonlinearities of the bridges are taken into consideration and both the equilibrium and the shape iteration are also carried out. Based on the convergent initial shapes determined by the different procedures, the natural frequencies and vibration modes are then examined in detail. The authors showed that the convergent initial shape can be found rapidly by the two-loop iteration method, a reasonable initial shape can be determined by using the linear computation procedure, and a lot of computation efforts can thus be saved. There is only small differences in geometry and prestress distribution between the results determined by linear and nonlinear computation procedures. However, for the analysis of natural frequency and vibration modes, significant differences in the fundamental frequencies and vibration modes will occur, and the nonlinearities of the cable-stayed bridge response appear only in the modes determined on basis of the initial shape found by the nonlinear computation.

2.2.2 Seisimic effects

R.A.Khan, et al. [4] in their work has presented using the concept of damage probability matrix. It is modeled as two dimensions system with deck idealized as a continuous beam subjected to bending action and axial compression. The response of the bridge is obtained by the frequency domain spectral analysis. The damage probability matrix is defined by three damages stated like major, moderate and minor. The seismic risk index is obtained by combining the damage probability matrix with the probability of occurrence of different magnitudes of earthquake. The earthquake intensity the expected peak values of stress at a critical section of the bridge deck and their distribution are determined. It is assumed that the bridge becomes unserviceable if the stress at the critical section of the bridge deck exceeds the threshold value and a notional failure of the bridge takes place. The annual probability of occurrence of different damage states are obtained but finding the probability of the peak stress exceeding a certain value. Thus a damage probability matrix is determined and annual probability of occurrence of earthquake intensities. The most reliable way of predicting the degree of system damage due to earthquake is the statistical analysis of the damage data of past earthquakes. The degree of damage of a cable stayed bridge is classified as a major damage, moderate or minor damages on comparison of its stress with respect to a specified barrier level. The mode shapes and the frequencies of the bridge are obtained using dynamic stiffness formulation. The deflection of the tower at the cable joint can be obtained by assuming that the tower behaves like a vertical beam fixed at the bottom end and constrained horizontally at the level of bridge deck and subjected to the transverse forces at the cable joints. The seismic input is assumed to be a Gaussian stationary process with zero mean and the system in linear. Annual probability of failure increases as degree of correlation between the ground motion at the support decreases.

Said M. Allam, T.K. Datta [5] has discussed in their work about frequency domain spectral analysis is presented for the seismic analysis of cable-stayed bridges for the multi-component stationary random ground motion incident at an angle with the longitudinal axis of the bridge. The ground motion is represented by its power spectral density function and a spatial correlation function. The analysis duly takes into account the spatial variation of ground motions between the supports, the modal correlation between different modes of vibration and the quasi-static excitation. The author uses the proposed method for analysis. Then parametric study is conducted to investigate the behavior of the cable-stayed bridge under the seismic excitation. The parameters include the spatial correlation of ground motion, the angle of incidence of the earthquake, the ratio between the three components of the earthquake, the number and nature of modes considered in the analysis, the inertia ratio between the tower and the deck, and the nature of the power spectral density function of the ground motion

2.2.3 Dynamic Effect due to Live Load

Raid Karoumi [**6**] in his paper has discussed method for modeling and analysis of cable stayed bridges under the action of moving vehicle is proposed. Bridge structure is analyzed using finite element method. Beam element method for modeling the girder and the pylonsis carried out. Two node catenaries cable elements are used for modeling cables. This method uses equivalent modulus approach. The vehicle used is suspension model of both primary and secondary suspension system. The dynamic response is evaluated using the mode superposition technique. Large deflections and vibrations caused by dynamic tire forces of the heavy vehicles leads to bridge deterioration. Dynamic response of the bridge to the moving vehicles is complicated as it has greater influenced by the interaction between vehicle and the bridge structure. Dynamic amplification factor is traditional used by the engineers as per the code. This factor is a function of natural frequency or span. But it is a conservative method as for some bridge but for other bridge it may underestimate the effect. In modern bridges major sources of damping has been eliminated and thus the damping is reduced to very low levels. For very long span cable-stayed bridge energy dissipation is very less and so it suppresses its own damping so to increase that external dampers are incorporated. Here a linear dynamic response of simple two dimensional cable bridge with moving vehicle is studied. The influence of vehicle speed, bridge damping, bridge-vehicle interaction and a tuned mass damper on bridge dynamic response is studied. As cable stayed are flexible so they exhibit geometrical non-linearity. In equivalent stiffness approach results in softer cable response as it accounts of sag effect but do not consider the stiffening effect due to large displacements. The external force vector is time dependent, bridge displacement, velocity and acceleration. The response increases with the increase in vehicle speed.

D. Bruno et al[**7**] has explain in their paper about the dynamic response of long span cable-stayed bridges subjected to moving loads. Here the analysis is based on a continuum model of the bridge, in which the stay spacing is assumed to be small in comparison with the whole bridge length. As a consequence, the interaction forces between the girder, towers and cable system are described by means of continuous distributed functions. A direct integration method to solve the governing equilibrium equations has been utilized and numerical results, in the dimensionless context, have been proposed to quantify the dynamic impact factors for displacement and stress variables. The authors present results with respect to eccentric loads, which introduce both flexural and torsional deformation modes. Sensitivity analyses have been proposed in terms of dynamic impact factors, emphasizing the effects produced by the external mass of the moving system and the influence of both "A" and "H" shaped tower configuration on the dynamic behaviour of the bridge is done.

2.2.4 Experimental and case study

Wei-Xin Ren et al[**8**] have performed both analytical and experimental studies on the Qingzhou cable-stayed bridge in Fuzhou, China and have published their work in technical paper an analytical modal analysis is performed on the developed three-dimensional finite element model starting from the deformed configuration to provide the analytical frequencies and mode shapes. The field ambient vibration tests on the bridge deck and all stay cables were conducted just prior to the opening of the bridge. The output-only modal parameter identification is then carried out by using the peak picking of the average normalized power spectral densities in the frequency-domain and stochastic subspace identification in the time-domain. A good correlation is achieved between the finite element and ambient vibration test results. It is demonstrated that the analytical and experimental modal analysis provide a comprehensive study on the dynamic properties of the bridge. The validated finite element model with respect to ambient vibration test results can serve as the baseline for a more precise dynamic.

K.V.Ramana Reddy et al. [9] in their research work found that all structures are subjected to cyclic loads like earthquake, blast loading etc which induces time varying stress in the materials. When the stress increases to larger value it fails. Fatigue is a process of progressive permanent internal structural changes in a material subjected to repetitive stresses. It is represented as number of cycles required to fail the material under a give repetitive stresses. There are two categories they are low-cycle loading or high stress loading which have few cycles but large stress ranges like seismic, wind stresses. The other type is high-cycle loading or low stress loading which have many in millions of cycles but at low stress ranges like traffic, ocean waves loading. The literature review has been done by the authors on various works done by researchers on the fatigue investigation of various materials. The damage done by high cycle loading is estimated by Stress-Number of cycles of failure (S-N) curve. Ti relates the applied stress range to the number of cycles to fracture of metal. The S-N curve is used in the Palmgren-Miner's rule to calculate the fatigue damage of a joint. The fatigue life is estimated by the equation given by Mallet giving number of cycles required to cause failure of a structural component subjected to stress range. Some fatigue rules are also discussed by the authors. A case study was carried out on "Ganaga Bridge" and compared the values of alternating stress in the reinforced components and the cable stays. The total life of the bridge is assessed by summation formulae of Miner. The fatigue lives of reinforcements are found out. Then the fatigue life of bent up bars for dead load and class AA loading was obtained and similarly for bent up bar and for cable stays of locked-coil steel wire rope type was used. The authors from their study derived some conclusions like, stirrups crossed by diagonal crack will resist higher stresses than design values, the higher grade of flexural steel does not prolong fatigue life, fatigue stress depends on alternating stress and not on maximum stress, and the fatigue strength of the locked wire rope is less than the maximum amplitude of stress.

2.2.5 Design of Components

Swami Saran [**17**]:The book covers analysis and design of pile foundation as per IS 456-2000.Pile cap are designed using bending theory and piles are designed as column considering one end fixed and other hinged.

Varghese .P.C [**16**]: It covers various geotechnical aspects for the design of foundation. It also gives various correlation of modulus of elasticity with SPT of soil. Concept of bearing capacity and settlement of raft and pile are explained.

Chapter 3

General Details of Cable Stayed Bridge

3.1 Configuration of cable stayed bridge

Here a brief overview of the structural configuration and the load resisting mechanisms of cable-stayed bridges is given. This is necessary because they are in many ways distinctly different from beam-type bridges and these differences strongly affect their behavior under static as well as dynamic loads. Cable stayed bridge can have a large variety of geometrical configuration; limited only by the creativity of the designer. The layout of the cable stays, the style of the pylons and the type of superstructure can be easily adjusted to suit the engineering requirements and to enhance the architectural beauty.

3.2 Components of Cable Stayed Bridge

a. Stiffening girder or deck system.

b. Cable system.

c. Towers or Pylons

3.2.1 Stiffening Girder or Deck system

The stiffening girder is supported by straight inclined cables which are anchored at the towers. These pylons are placed on the main piers so that the cable force can be transferred down to the foundation system as shown in the Fig 3.1.

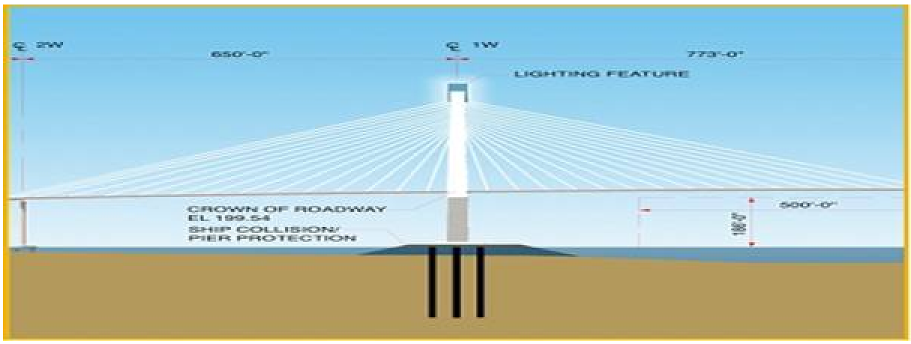


Figure 3.1: Cable stayed bridge elevation

Close supporting points enables the deck to be slim. It is loaded mainly in compression with the largest prestress being at the intersection with the towers. This is due to the horizontal force which is applied by each of the cables.

The role of the stiffening girder is to transfer the applied loads, self weight as well as traffic load, onto the cable system. These girders have to resist considerable axial compression forces besides the vertical bending moments. This compression force is introduced by the inclined cables. The girder can be either of concrete or steel. For smaller span lengths concrete girders are usually employed because of the good compression characteristics. However, as the span increases the dead load also increases, thus favoring steel girders. The longest concrete bridge that has been constructed is the Skarnsund Bridge, Norway, with a main span of 530 m. There are three types of girder cross sections used for cable-stayed bridges

a. Longitudinal edge beams

b. Box girders.

c. Trusses.

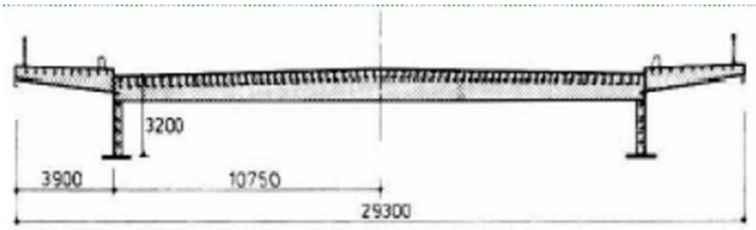


Figure 3.2: longitudinal beam arrangement

Figure 3.3: Typical Box Girders

Figure 3.4: Typical Truss girder

3.2.2 Cable Systems

Cable patterns The cable system connects the stiffening girder with the towers. There are essentially four patterns which are used.

- Fan type.
- Harp type.
- Mixed type.
- Star type.

Fan type

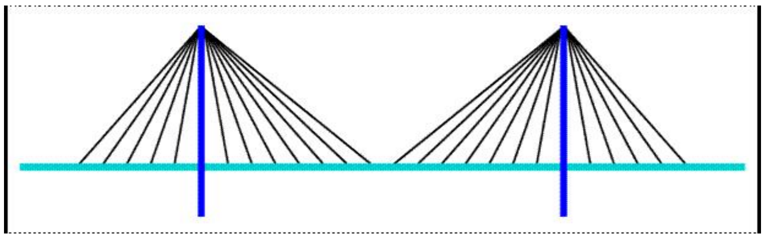


Figure 3.5: Fan type arrangement

Figure 3.6: Albert Bridge, London fan type arrangement

The fan type is more aesthetic and is most economical for a pylon of slenderness ratio (h/l) < 0.29. For an equal tower height the average inclination of the cable stays is lower. However the cable stays are longer and converge towards a single point at the top of the tower, they pose problems of anchoring arrangements and any subsequent stay replacement is difficult. The fan type increases the buckling problem due to greater effective strut length in the towers as in Fig. 3.5 and 3.6.

Harp type

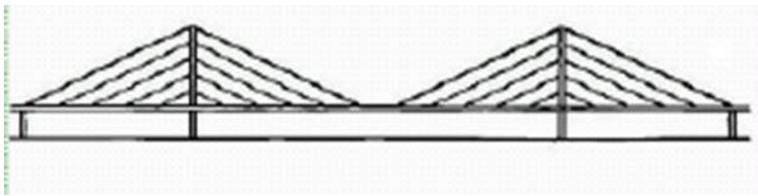


Figure 3.7: Typical Harp type arrangement

Figure 3.8: Abdoun Bridge, Amman, Jordan

The harp type of system is preferred in a double plane system as it reduces the intersection of cable when viewed from a oblique angle as in Fig. 3.7 and 3.8. In the harp type of system cable connections are distributed throughout the height of the tower and hence results in an efficient tower design in comparison with fan type. The structural behavior of the tower varies depending upon the type of cable system.

Mixed type or Semi Harp

Figure 3.9: Typical arrangement of Semi Harp type arrangement

Figure 3.10: The Okutama Cable-stayed Bridge,Tokoyo,Japan

The mixed type is used when it is difficult to accommodate all cables at the top of the tower as in Fig. 3.9 and 3.10.

Star type arrangement

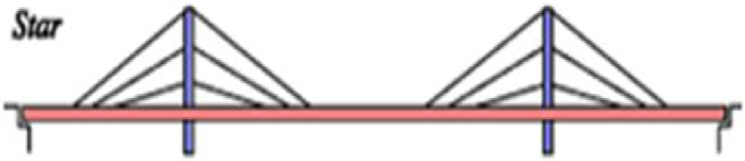


Figure 3.11: Typical arrangement of star type

Figure 3.12: Ormiston Road Bridge by Moller Architects in New Zealand

Star types system is preferred for its unique aesthetic appearance. It becomes the landmark of that place as in Fig. 3.11 and 3.12.

Type of cables

The success of cable-stayed bridges in recent years can mainly be attributed to the development of high strength steel wires. These are used to form ropes or strands, the latter usually being applied in cable-stayed bridges.
There are four types of strand configuration

- Parallel wire strand.
- Helically-wound strand.
- Locked coil strand.
- Structural Rope.

Parallel wire strand

Parallel wire strands consist of galvanized round wires laid up in a hexagonal form, with a long helix. The product is then encased in a tight fitting High Density Poly Ethylene tube as in Fig 3.13. Parallel wire bundles are ideal for cable stayed bridges, offering high axial stiffness and an especially high modulus

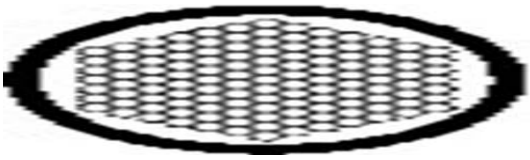



Figure 3.13: Typical cross section of parallel wire stands

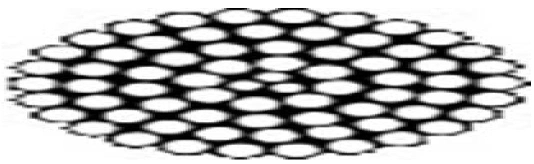



Figure 3.14: Typical helical wounded strands

Spiral strands consist of large diameter galvanised round wires helically spun together. The wires are stranded in one or more layers, mainly in opposite directions, to achieve the required diameter. This is the most versatile structural cable, with high breaking strength and a good strength to weight ratio.

Locked coil strand

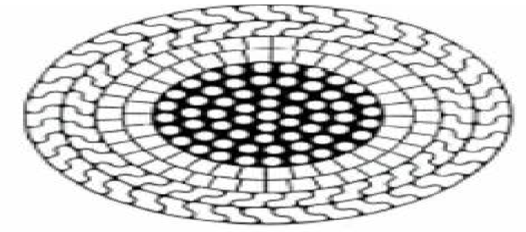


Figure 3.15: Typical cross section of locked coil strands

Locked coil strands consist of a centre of one or more layers of large diameter galvanized round wires helically spun together as shown in fig 3.15 . Stranded on top of this centre are one or more layers of large diameter galvanized shaped wires, mainly in opposite directions, to achieve the required diameter. The closed construction and smooth outer layers offer high resistance to deformation and specific pressures.

Structural Rope

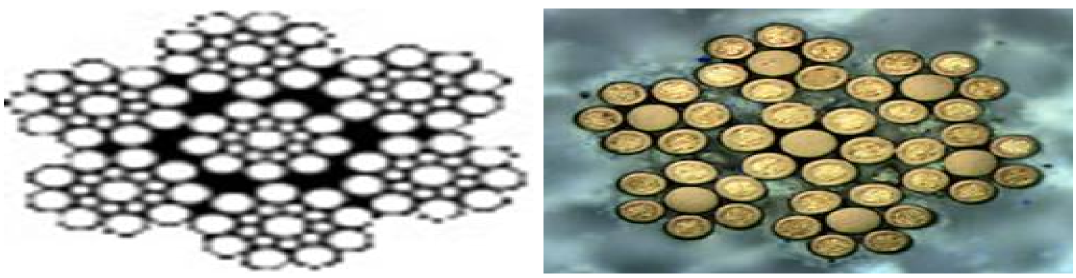


Figure 3.16: Cross section of structural rope

Structural ropes are usually of 6 strand construction helically spun around a steel core, comprising smaller diameter galvanized wires giving a high degree of flexibility. Structural ropes have a low modulus and are unsuitable for most structural applications as shown in the Fig 3.16. However, they offer an economic solution for temporary structures, and where greater flexibility is required.
The application of the various cable strands which are preferred with various different types of cable bridges are as discussed in the below shown Table. 3.1

Table 3.1: Principal Applications

	Cable Type			
Bridge Design	**Spiral Strand**	**Locked Coil Strand**	**Structural Rope**	**Parallel Wire Strands**
Suspension Bridge	Main Cables Barriers/Parapets Hand Strand Hangers/Suspenders		Hangers/ Suspenders Temporary Supports	
Cable Stayed Bridge	Main Stay Bracing Cables Barriers/Parapets	Main Stays	Vibration Damping Systems Temporary Supports	Main Stays
Tied Arch	Hangers/Suspenders	Hangers/ Suspenders		

The properties of the various cable strands with minimum and maximum breaking forces and their minimum and maximum nominal diameter are as discussed in the below shown Table. 3.2

Table 3.2: Typical Range of Properties

Cable	**Minimum Nominal Diameter (mm)**	**Minimum Breaking Force (kN)**	**Maximum Nominal Diameter (mm)**	**Maximum Breaking Force (kN)**
Spiral strand	13	171	150	20800
Locked coil strand	30	858	150	21500
Structural rope	13	102	102	6492
	No of wires		No of wires	
Parallel wire strand	73	4400	421	27060

Anchorage of cables

Cables need to be anchored at the deck as well as at the towers. For each of these connections numerous devices exist depending on the configuration of deck and tower as well as of the cable.Some arrangements for tower and deck are shown in Fig. 3.17 and Fig. 3.18 respectively.

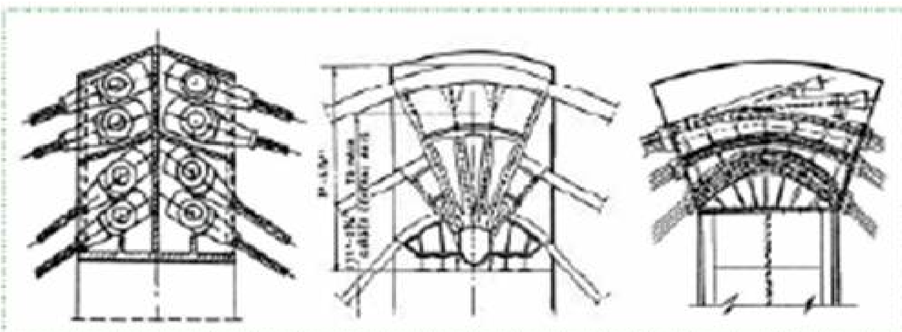


Figure 3.17: Cable anchorages in tower

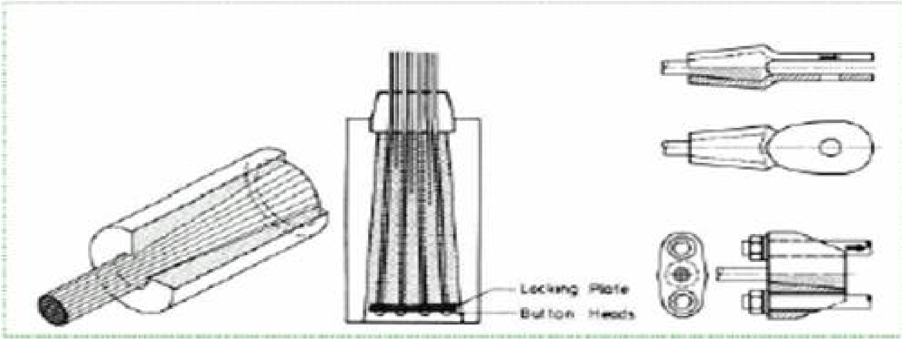


Figure 3.18: Devices for cable anchorages with deck

Cable supports at the tower may be either fixed or movable. They are situated at the top or at intermediate locations mainly depending on the number of cables used. While fixed supports are either by means of pins or sockets, movable supports have either roller or rocker devices. Connections to the deck are by means of special sockets. Their configuration strongly depends on the type of cable used. Usually these sockets are threaded and a bolt is used to allow adjustments on the tension of the cable.

3.2.3 Tower or pylons shapes

The function of the towers is to support the cable system and to transfer its forces to the foundation. They are subjected to high axial forces and they are also subjected to bending moments as well, depending on the support conditions. It has already been pointed out that the towers in harp-type bridges are subjected to severe bending moments. Box sections with high wall widths generally provide best solutions. They can be kept slender and still possess high stiffness. Towers can be made of steel or concrete. Concrete towers are generally cheaper than equivalent steel towers and have a higher stiffness. However, their weight is considerably higher and thus the choice also depends on the soil conditions present. Furthermore, steel towers have advantages in terms of construction speed. The tower is the principal compression member transmitting the load to the foundation. Tower is of different types to accommodate different cable arrangements, bridge site condition; design features aesthetics and economical considerations. Generally the arrangement of the cables determines the design of both pylon and deck. The various possible types of tower construction are illustrated in figures below which shows that they may take the form of:

a. H-Type or Twin Tower

b. Y-type tower

c. Inverted Y- Type tower

d. Delta Tower

e. Diamond Tower

Figure 3.19: Typical H-type pylon

Figure 3.20: Y-type pylon

Figure 3.21: Inverted V-type pylon

Figure 3.22: Delta type towers

Figure 3.23: Diamond type arrangement of pylon

3.2.4 Advantage and Disadvantages of cable stayed bridge

Advantages:

- Efficient use of materials.

- Faster speed of erection.
- Reduced vertical deflection compared to suspension bridges.
- Superstructure is light.
- Low superstructure depth-to-span ratio.
- Maintenance cost is reduced.
- Aesthetic compatibility in many locations.
- Uncomplicated structure, resulting in ease of fabrication or casting
- Structural behavior cable stayed systems occupy a middle position between the girder type and suspension type bridge.
- While constructing the deck cantilevering out from the tower, the cables act both as temporary and permanent supports to the bridge deck.
- The girders take horizontal compressive forces due to the cable action and no massive anchorages are required so substructure is economical.
- It's geometrically unchangeable under any load position on the bridge, and all cables are always in a state of tension so light flexible members can be used.
- For a symmetrical bridge, the horizontal forces balance and large ground anchorages are not needed.
- This type of bridge can be constructed with single tower also, while the suspension bridge has to be constructed with two towers.

Disadvantages:

- Susceptible to wind induced oscillations
- Corrosion of cables due to weathering action.
- Due to low center of gravity, sinking of ground occurs during earthquakes.

3.3 Structural anchorages

The axial force in the stiffening girder depends upon the method of anchoring the cables and the provision of expansion joints and their location in the structure. Basically three different types of anchored systems are considered such as:

a. Self anchored system.

b. Fully anchored system.

c. Partially anchored system.

3.3.1 Self anchored system

In this self anchored system, there is no restraint at the supports to the horizontal components of the cable force. In this case the axial force distribution in the girder will vary from zero at the centre of main span to maximum compression near the tower.

3.3.2 Fully anchored system

In this type, no provision is made for the movement at the supports but expansion joints are provided at the towers. The axial force distribution changes with zero force at towers increasing to a maximum value at the centre of span.

3.3.3 Partially anchored system

In the partially anchored system the axial forces are considerably reduced using the combination of above two systems by providing horizontal restraint at the abutments with no expansion joints or expansion joints provided only in the end spans.
The anchorage must be designed to allow adjustment of length and replacing a cable damaged by an accident without interrupting the traffic. They must be futher designed to prevent bending stress in the wires or strands at the socket due to change

of sag or due to slight oscillations of the cables caused by the wind eddies. At the tower head, cables running over a saddle like a suspension bridge should be avoided because their replacement can be difficult. It is preferred to anchorage the cable at each side individually and design suitable devices to carry the horizontal component of the cable forces through the tower. At the top of the towers, the box must be wide enough to allow access and handling of the jacks.

3.4 Verification example

For verification of the software SAP2000 V 14 used for the analysis of the cable stayed bridge and to validate the proper methodology used for preparation of model in the SAP2000 a data from a technical paper has been taken as a validation problem.

Data Single pylon of cable stayed bridge with asymmetrical span as shown in the Fig 3.24

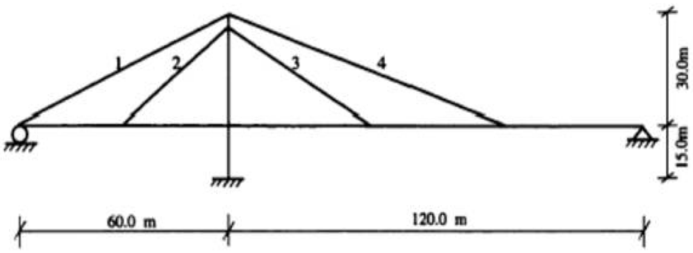


Figure 3.24: Elevation of cable stayed bridge

Span 1:	60m.
Span 2:	120m.
Height below deck	15m.
Height above deck	30m.
Deck slab dimension	
Width:	15m.
Depth:	1m.
Modulus of elasticity:	3.5 x $10^4 kN/m^2$.
Poisson's ratio:	0.1666
Pylon dimension:	
Shape:	H
D:	6.326m.
B:	1m.
Cable property	
Modulus of elasticity:	2 x 10^5 M Pa.
Specific weight:	$78kM/m^3$.
Area	
cable 1	$0.004m^2$
cable 2	$0.002m^2$
cable 3	$0.002m^2$
cable 4	$0.004m^2$

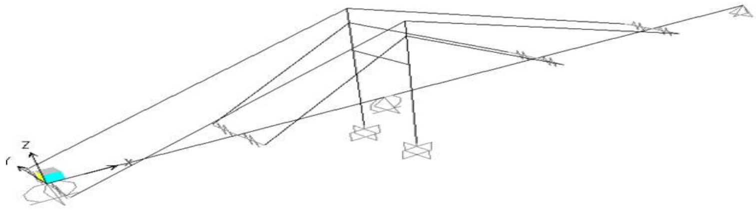


Figure 3.25: Cable stayed bridge model in SAP2000

The cable stayed bridge model is prepared in SAP2000 as shown in the Fig. 3.25 above. The natural frequency from the paper as in table 3.3 is compared with the results from the software package which are discussed below as in snapshot of SAP 2000 in Fig 3.26.

OutputCase Text	StepType Text	StepNum Unitless	Period Sec	Frequency Cyc/sec	CircFreq rad/sec	Eigenvalue rad2/sec2
MODAL	Mode	1	3.725936	0.26839	1.6863	2.8437
MODAL	Mode	2	1.728482	0.57854	3.6351	13.214
MODAL	Mode	3	1.645203	0.60783	3.8191	14.585
MODAL	Mode	4	1.54724	0.64631	4.0609	16.491
MODAL	Mode	5	0.997041	1.003	6.3018	39.713
MODAL	Mode	6	0.807887	1.2378	7.7773	60.486
MODAL	Mode	7	0.780835	1.2807	8.0468	64.75
MODAL	Mode	8	0.595755	1.6785	10.547	111.23
MODAL	Mode	9	0.472417	2.1168	13.3	176.89
MODAL	Mode	10	0.432664	2.3113	14.522	210.89

Figure 3.26: Time period from SAP2000

Table 3.3: Table of Comparison

MODE No.	**Frequency (Cyc/sec)**		
	Paper results	**Author's results**	**SAP2000 results**
1	0.251	0.249	0.268
2	0.554	0.564	0.578
3	0.723	0.742	0.608
4	1.029	1.2	0.646
5	1.228	1.239	1.003
6	1.535	1.432	1.239
7	1.756	1.664	1.281
8	1.859	1.8	1.678
9	2.031	1.909	2.116
10	2.134	2.163	2.311

The comparison of frequency of the author's 1 results are explained in the Table 3.3. The the characteristic mode shape of the cable stayed bridge are shown as in the Fig. 3.27 and 3.28 the mode shapes here shown are modes 1, 3, 7 and 10

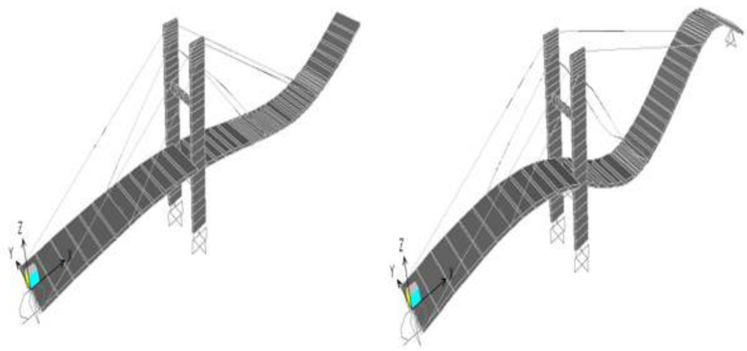


Figure 3.27: Mode 1 and Mode 3

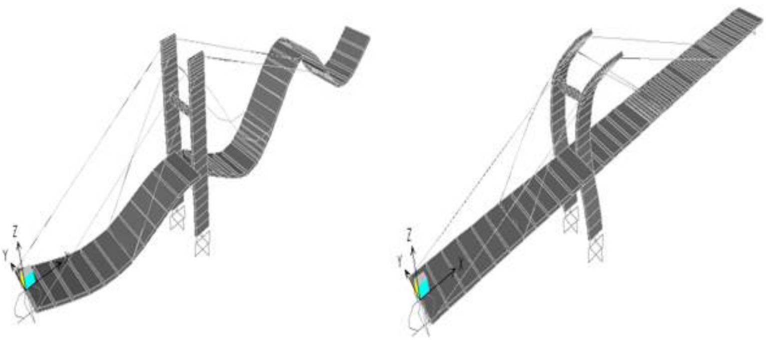


Figure 3.28: Mode 7 and Mode 10

Chapter 4

Analysis of Cable Stayed Bridge

4.1 Introduction

SAP2000 is a powerful and versatile tool for analysis and design of structures based on static and dynamic finite element analysis. Non linear analysis can also be performed in SAP2000. The analytical capabilities are just powerful representing the latest research in numerical techniques and solution algorithms. The program is structured to support wide variety of the latest national and international codes for both steel and concrete designs Backup design information procedure by the program is also provided for convenient verification of the results. English as well as SI and MSK units can be used to define the model geometry and to specify different parameters.

4.2 Finite Element Modelling

4.2.1 Deck

A box girder of concrete is shown in the Fig. 4.1 It has to be modeled such as to behave correctly in bending and torsion and to resemble the inertia effects correctly. The finite element modeling is shown in Fig. 4.2 A modeling approach is used in which a single central spine of linear elastic beam elements that has the actual bending and torsional stiffness of the deck. Each of spine element beams has a length of 10m connected by cables at each ends. At these nodes rigid links are used to connect with the cable in perpendicular direction to the beam element. This is done to achieve the proper offset of the cables with respect to the centre of inertia of the spine. Using the rigid link option allows the displacement of all nodes in same manner, thus not introducing additional degree of freedom into the model. Bridge deck section a box girder is modeled as a straight frame element, the cross section of the box girder as show in the figure is made in the section designer of the SAP2000.

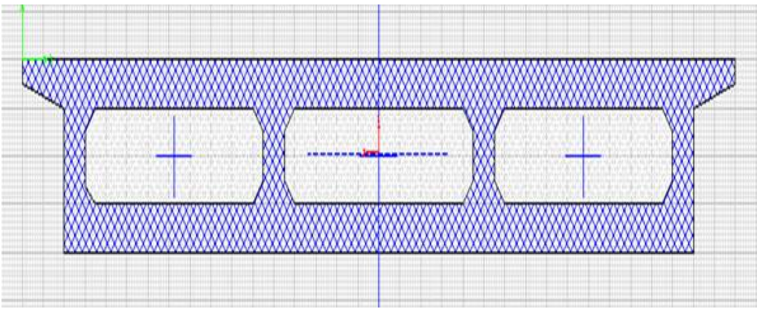


Figure 4.1: Cross section of bridge deck in SAP2000

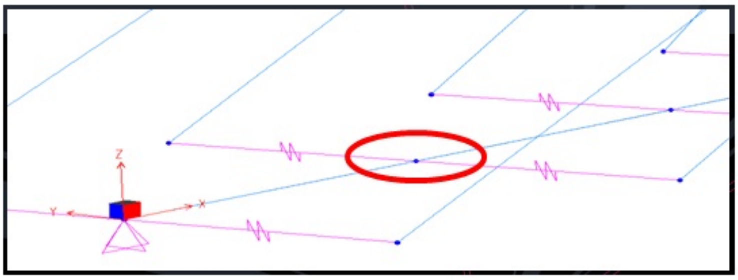


Figure 4.2: Finite element of bridge deck

4.2.2 Cables

The modeling of cables is a difficult issue because of non-linearities arise from the cable sag. The stiffness thus changes when load is applied. However a linear frame element is utilized for modeling of cables. Taking into account the cable sag and thus non linear cable behavior by means of equivalent stiffness would have been extreme tedious since every cable would have to associated with a different force-displacement relationship because of the changing inclination and length of the cables. This would have also considerably increase the computation time.

The prestress was applied to all the cables in order to ensure a small deformation of the deck when the self weight is applied. The bridge was modeled picking up the geometry from the design drawings. Since these show the as built configuration the application of the self weight to the structure has to be taken into account. In reality the cables are prestress according to a prior calculation so that the final shape is correct. Accordingly here prestress is adjusts so as to have as small as possible deflection when the self weight is applied.

Pretensioning of Cables.

A cable is converted in to a rod model, with its sectional area alone being considered. The bending rigidity of the cable is ignored. Converted modulus of elasticity Eeq by the equation of H. J. Ernst, as shown below is used to consider reduction of rigidity due to influence of cable sag. where

$$E_{eq} = \frac{E}{[1 + \frac{(wH)^2 AE}{12T^3}]} \tag{4.1}$$

E_{eq} = Equivalent modulus of elasticity of the cable.

E. = Modulus of elasticity of the straight cable (2 x 10^5 MPa).

W = Weight of the cable per unit length.

A = Cable sectional area.

H = Horizontal projected length of the cable.

T = Cable initial tension.

Cable Initial Tension

The main objective of providing initial tension to cables is to minimize the bending-moment and deflection of the deck under dead loads. The method used to determine the initial cable tension is Simple beam method which is described below.

a. Calculate the effective span (S) for each cable at the deck.

b. Compute the total dead load (W) of each effective span. ($W_i = wx \sin\theta$, w is the weight per unit length).

c. Compute the inclined angle between the cable and the bridge deck at the cable connection ($\theta_i = \tan^{-1}\frac{h_i}{l_i}$).

d. Compute the cable axial force at the cable lower end by using the results of steps-2 and 3 ($P_i = \frac{W_i}{\sin\theta}$). Check the cable prestrains.

By applying the initial tensions the vertical displacements of the bridge deck and the horizontal movement of the top of the pylons can be reduced to @80%. In addition the initial cable tension increases the moment carrying capacity of the deck. The cable areas, pretensions and Equivalent modulus are different for different spans. Here Table 4.1 these values are given for 200m H-Frame Bridge for example. Areas are in m^2, cable tensions are in k N and E_{eq} are in kN/m^2.

Table 4.1: Initial cable forces

CABLE NOS	C/S AREA m^2	INITIAL FORCES(k N)	E_{eq} kN/m^2
1	0.004	3828.99	199946745.01
2	0.004	3538.79	199953150.88
3	0.003	3218.18	199983178.59
4	0.003	2869.95	199986658.62
5	0.002	2509.30	199997371.35
6	0.002	3278.53	199999705.36

4.2.3 Pylon

Pylon modelling is done as beam elements. The pylon legs unify at the pylon head which was divided into beam elements as long as the distance between the cable anchorage points, thus readily allowing for the connection of the cable elements.

4.2.4 Support Condition

There are different boundary conditions prevailing for the connection of the frame element to each other i.e. girder-pylon connection, girder-cable connection etc. Here in the study beam on elastic support analogy has been considered.

4.2.5 Loading Condition

Moving load analysis in the present work is performed using IRC classA and IRC class70R vehicular live loads. The vehicles are generated and applied in the existing lanes following the guidelines from IRC:6 (2000).The following codal provisions are followed in the present work regarding the live loads.

Live Load *(I.R.C:6-2000 Clause No. 207)* Road bridges and culverts shall be divided into classes according to the loading as they are designed to carry. *(I.R.C:6-2000 Clause No. 201).* The *I.R.C:6-2000* specifies three classes of loads, designated as class 70-R, class AA and class A for the design of permanent bridges.

Live load combination *(I.R.C:6-2000 Clause No. 207.4)* as shown in the table 4.2

Table 4.2: Live load combination

Carriageway width	**Number of lanes**	**Load combination**
Less than 5.3 m	1	One lane of Class A considered occupying 2.3m.

		The remaining width of carriageway shall be loaded with 500 kg/m^2
5.3m and above but less than 9.6 m	2	One lane of Class 70R OR two lane of class A
9.6m and above but less than 13.1 m	3	One lane of Class 70R for every two lanes with one lane of class A on the remaining lane OR 3lane of class A
13.1m and above but less than 16.6 m	4	One lane of Class 70R for every two lanes with one lane of class A for the remaining lanes, if any, OR one lane of class A for each lane
16.6m and above but less than 20. M	5	
20.1m and above but less than 23.6 m	6	

So in this problem as we have four lanes and length of carriage width of 17m we here take the combination as one lane of Class 70R for every two lanes with one lane of class A for the remaining lanes, if any, OR one lane of class A for each lane.

Reduction in the longitudinal effect on Bridges accommodating more than two traffic lanes (I.R.C:6-2000 Clause No. 208) Reduction in the longitudinal effect on bridges having more than two traffic lane due to the low probability that all lanes will be subjected to the characteristic loads simultaneously shall be in accordance with the

Table 4.3.

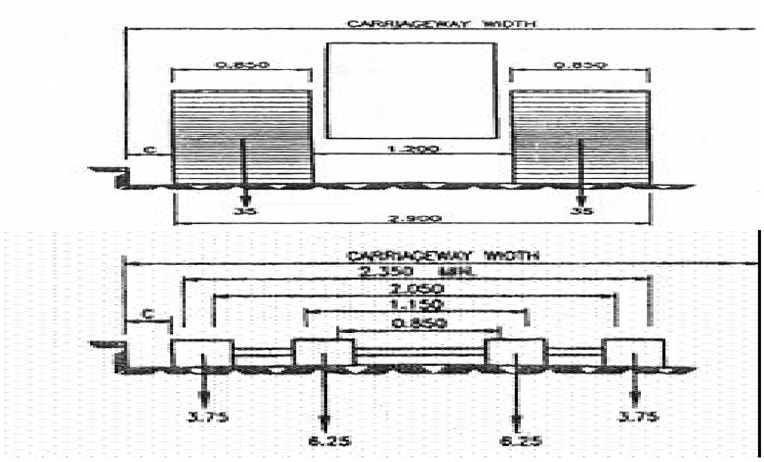
Table 4.3: Reduction in Live Load

Number of lane	Reduction in longitudinal effect
For two lane	No reduction
For three lane	10% reduction
For four lane	20% reduction
For five or more lane	20% reduction

So here in this problem as there are four lanes so we introduce 20% reduction in longitudinal effects.

Class AA loading

This loading is to be adopted with certain municipal limits, in certain existing or contemplated industrial areas, in other specified areas and along certain specified highway. Bridges designed for Class AA as in Fig. 4.3 loading should be checked for Class A loading also, as under certain conditions, heavier stresses may be obtained under Class A loading. Where Class 70-R is specified, it shall be used in place of Class AA loading.



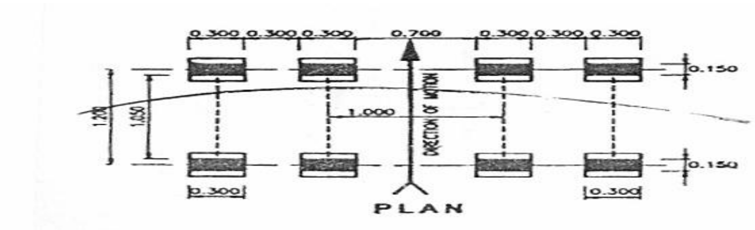


Figure 4.3: Class AA: Tracked and wheeled vehicles

Class A loading This loading is to be normally adopted on all roads on which permanent bridges and culverts are constructed as in the Fig. 4.4

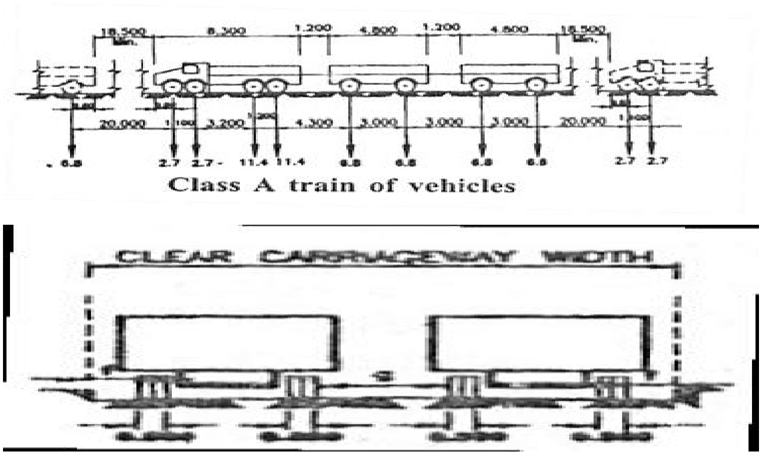


Figure 4.4: Class A: Wheeled vehicles

Impact force: ***(I.R.C:6-2000 Clause No. 211)***

Provision for impact or dynamic action shall be made by an increment of the live load by an impact allowance expressed as a fraction or a percentage of the applied live loads.

For Class A or Class B loading:

In the members of any bridge designed either for Class A or Class B loading, impact percentage shall be determined from the curve shown in Fig.4.5 The impact factor shall be determined from the following equations, which are applicable for spans between 3 m. and 45 m.

$$(i) Impact factor for R.C. bridges = \frac{6+L}{4.5} \tag{4.2}$$

$$(ii) Impact factor for steel bridges = \frac{13.5+L}{9} \tag{4.3}$$

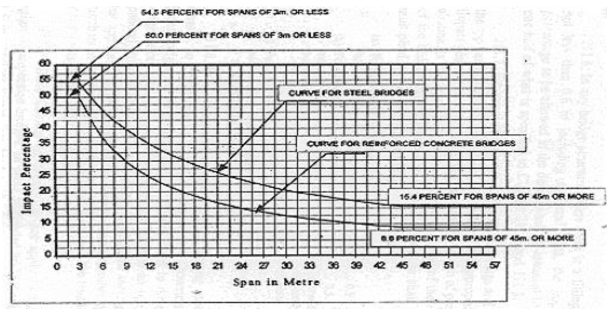


Figure 4.5: Impact % for highway bridges for class A and B Loading as per IRC:6-2000

For Class AA and Class 70-R loading:

The value of the impact percentage shall be taken as follows:

(a) For spans less than 9 m.

(i) For tracked vehicles: 25% for span upto 5 m linearly reducing to 10% for span of 9 m.

(ii) For wheeled vehicles: 25%

(b) For spans of 9 m or more.

(i) Reinforced concrete bridges:

Tracked vehicles: 10% up to a span of 40 m. and in accordance with the curve in Fig. 4.5 for spans in excess of 40 m.

Wheeled vehicles: 25% for spans up to 12 m. and in accordance with the curve in fig.4.16 for spans in excess of 12 m.

(ii) Steel bridges:

Tracked vehicles: 10% for all spans

Wheeled vehicles: 25% for spans upto 23 m and in accordance with the curve in Fig. 4.5 for spans in excess of 23 m.

No impact allowance shall be added to the footway loading.

Moving load generation in SAP2000

Moving load generation in SAP2000 consists of generating the traffic lanes and the vehicles which will run over these lanes.

Traffic lanes

Four lanes of traffic each 3.75 m wide are generated on top of the 17.0 m wide 3 cell concrete box girder.

The following menus should be accessed for creating the traffic lanes in a SAP2000 bridge model as in Fig. 4.6.

→ Define → Bridge loads →Lanes

The following dialogue-box will appear.

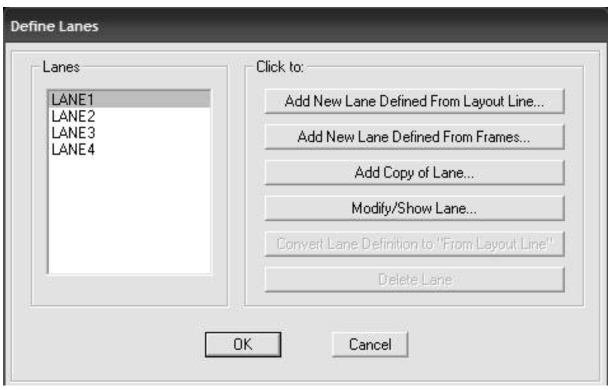


Figure 4.6: Define lane dialogue box

On clicking the add new lane following dialogue box appears as in Fig. 4.7.

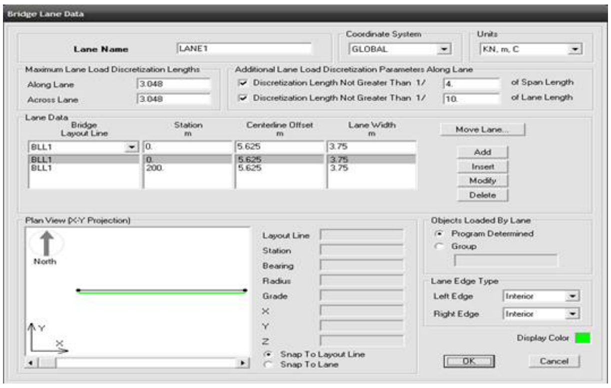


Figure 4.7: Bridge lane data dialogue box

The centre line of the lane is defined with respect to the layout line and the width of the lane is also shown in Fig. 4.8.

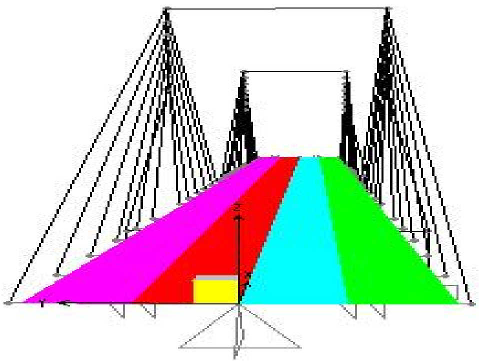


Figure 4.8: Lanes as seen in SAP2000 model

Vehicles load in SAP2000 IRC class A and class 70R wheeled vehicles are generated as the general vehicle definition form of SAP2000.

Access the following menus to generate general vehicles.

→ Define → Bridge loads → Vehicles.

The following dialogue-box will appear as in Fig. 4.9.

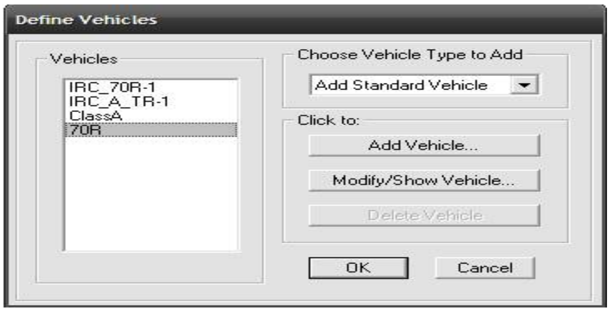


Figure 4.9: Vehicle define dialogue box

On adding new vehicle command below shown dialogue box appears as in Fig. 4.10.

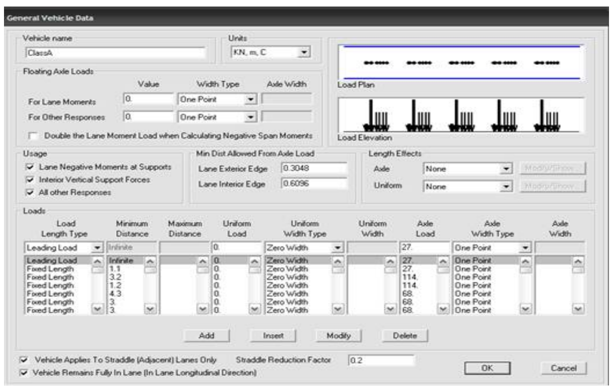


Figure 4.10: Vehicle data dialogue box.

Specify the distances between two successive axles, axle load and axle width of the vehicle. The figure above shows the class A vehicle. The distance between two successive vehicles is takes as 20.0 m as per *IRC: 6 (2000)*.

Moving load Application

The vehicles are applied to the lanes using the vehicle classes.

Vehicle classes

The designer is often interested in the maximum and minimum response of the bridge to the most extreme of several types of vehicles rather than the effect of the individual vehicles. The maximum and minimum force and displacement response quantities for a vehicle class will be the maximum and minimum values obtained for any individual vehicle in that class. Only one vehicle ever acts at a time.

All the vehicles are applied to the traffic lanes through the vehicle classes. If it is desired to apply an individual vehicle load, one has to define a vehicle class containing only that single vehicle.

A vehicle class can be created by accessing the following menus of SAP2000.

→ Define →w Bridge loads → Vehicle classes.

The following dialogue-box will appear as in Fig. 4.11.

Figure 4.11: Vehicle classes definition dialogue box.

On clicking add new class the below dialogue box appears as in Fig. 4.12.

Impact factor of 8.8% has been taken from the graph in Fig 4.5 given in IRC 6. This is added to the scale factor of the vehicle class data.

Moving load analysis cases

The final step in the definition of the vehicle live loading is the application of the vehicle class to the traffic lanes. This is done by creating independent moving-load analysis cases.

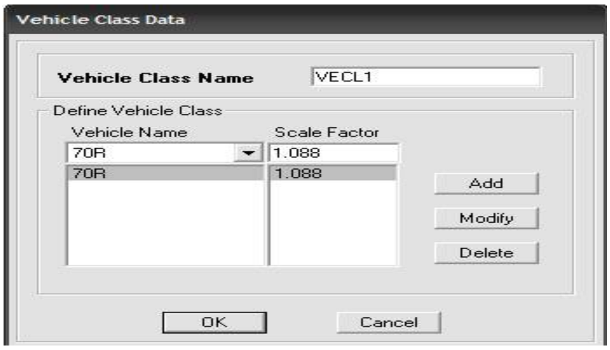


Figure 4.12: Vehicle class data dialogue box.

A moving load case is a type of analysis case. Unlike most other analysis cases you can not apply load cases in a moving load case. Instead each moving load case consists of a set of assignments that specify how the classes are applied to the lanes.
Access the following menus to reach to moving load analysis form.
→ Define → Load Cases
The following dialogue-box will appear as in Fig. 4.13 and 4.14.

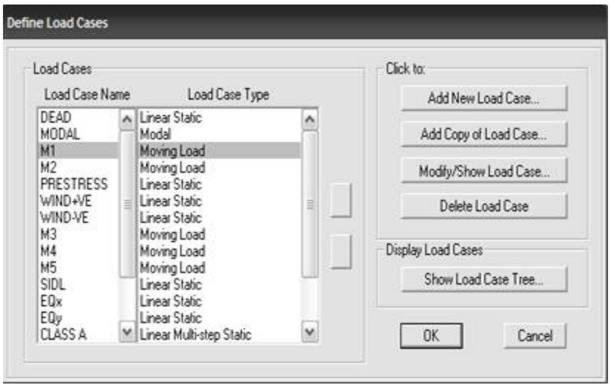


Figure 4.13: Load cases dialogue box

In the analysis case tab select Moving Load (M1). Here in this analysis case vehicle class VECL 2 containing class70R vehicle is applied to lanes 1 and 2, while the vehicle class VECL 1 classA containing vehicle is applied to the lane 3. Similarly other moving loads are also defined in same way with changing VECL 2 and VECL

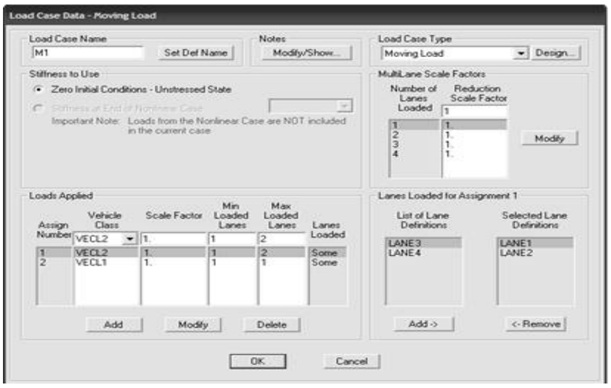


Figure 4.14: Load cases for moving load dialogue box.

1 as required as in Fig 4.13 and 4.14.

4.3 Structural Data

As shown in the Fig. 4.15 the cable arrangement of one side of the pylon is shown as it is symmetrical in both side. The Data for the Cable stayed bridge has been taken as discussed below.

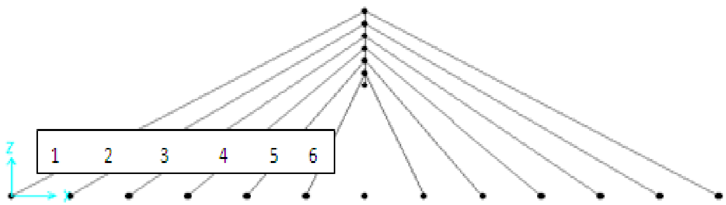


Figure 4.15: Cable arrangement of bridge

The 3-D rendered view in Fig. 4.17 and 3-D skeleton view as in Fig. 4.16 of the Cable stayed bridge is shown for better understanding of the structural data taken.

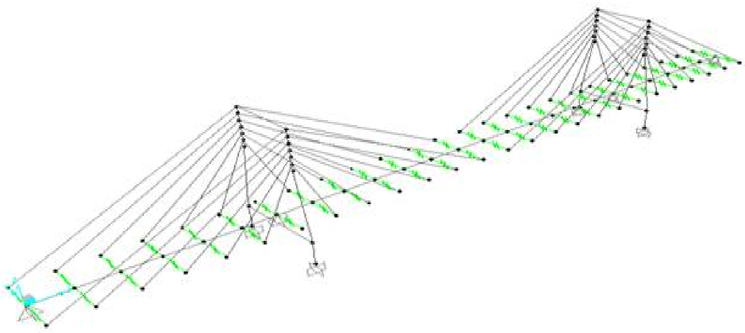


Figure 4.16: 3D skeletal form

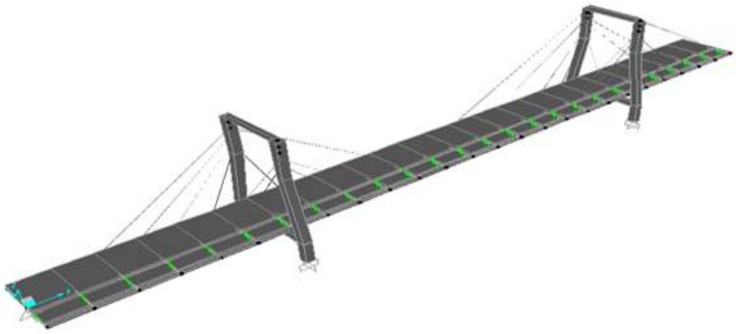


Figure 4.17: 3D rendered view

Central span	100m
Side span:	50m
Height below deck:	10m
Height above deck:	30m
Deck concrete box girder dimension	
Width:	17m
Depth:	2m
Modulus of elasticity:	33541 M Pa
Grade of concrete:	M45
Pylon dimension	
Shape:	H type
B:	2m
D:	2m

Top cross beams	
B:	1m
D:	1.5m
Bottom cross beams	
B:	1m
D:	2m
Cable properties	
Modulus of elasticity	2 x 10^5M Pa
Areas	
cable 1	$0.004m^2$
cable 2	$0.004m^2$
cable 3	$0.003m^2$
cable 4	$0.003m^2$
cable 5	$0.002m^2$
cable 6	$0.002m^2$

Chapter 5

Aerostatic and Dynamic Loadings

5.1 Aerostatic Analysis

Bridges are frequently exposed to severe wind conditions. Aerostatic loads on bridge superstructure depend on the type of bridges, such as truss, arch, cable-stayed or suspension. Other parameters that affect aerostatic loads on bridge superstructures are the wind velocity, angle of attack, the size and shape of the bridge, the terrain, and the gust factors. Here a general discussion on the aerostatic loads and their effect on the bridge structure has been presented.

Aerostatic loads form a major component of lateral loads that act on the structure. Because of the increasing span, cable-stayed bridges have become increasingly sensitive to aerostatic stability. Experimental observations suggest that the aerostatic instability occur under the action of static aerostatic loads. Therefore the aerostatic stability analysis of long span cable stayed bridge under the displacement-dependent loads is of considerable importance.

5.2 Aerostatic loads

The components of wind forces per unit span acting on the deformed deck can be written in wind axes as:

$$DragForceFy(\alpha) = \frac{1}{2}\rho Vz^2 Cy(\alpha)D \tag{5.1}$$

$$LiftForceFz(\alpha) = \frac{1}{2}\rho Vz^2 Cz(\alpha)B \tag{5.2}$$

$$PitchingMoment : M(\alpha) = \frac{1}{2}\rho Vz^2 Cm(\alpha)B^2 \tag{5.3}$$

Where Cy(α) Cz(α),Cm(α) is the coefficient of drag force, lift, force and pitching moment.

The wind forces above are the function of the torsional displacements of the structure. They vary as the girder displaces. Therefore the three components pf aerostatic loads are displacement dependent as shown in the Fig. 5.1

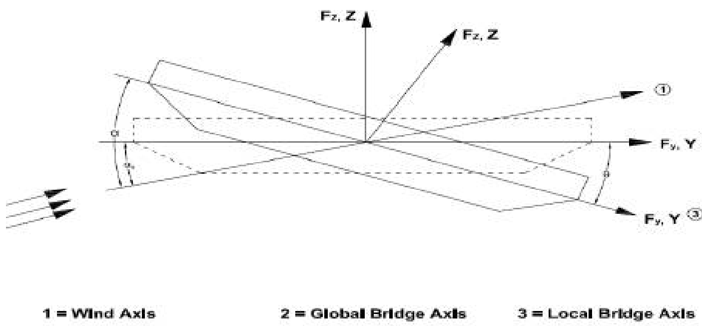


Figure 5.1: Motion of Bridge Deck and three Components of Wind loads in Different Axes

The above wind forces can be transformed to the wind forces in global bridge axis as:

where

$$F_y(\alpha) = \frac{1}{2}\rho V z^2 Cy(\alpha) D \tag{5.4}$$

$$F_z(\alpha) = \frac{1}{2}\rho V z^2 Cz(\alpha) B \tag{5.5}$$

$$M(\alpha) = \frac{1}{2}\rho V z^2 Cm(\alpha) B^2 \tag{5.6}$$

where

$$C_y(\alpha) = \left[C_D(\alpha) - \frac{C_L(\alpha) B \tan\alpha_0}{A_N} \right] \sec\alpha_0 \tag{5.7}$$

$$C_z(\alpha) = \left[C_L(\alpha) - \frac{C_D(\alpha) A_N \tan\alpha_0}{B} \right] \sec\alpha_0 \tag{5.8}$$

$$C_M(\alpha) = [C_L(\alpha)] \sec^2\alpha_0 \tag{5.9}$$

B is the deck width m.

D is the vertical projected area m.

ρ is the density of air kg/m^2.

V_z is the design wind speed m/s^2.

$V_r = V_z$ cosα

$V_z = V_b$ x K_1 x K_2 x K_3

K_1 = Risk coefficient cl 5.3.1.IS 875 part 3 18

K_2 = terrain height and structure size factor cl.5.3.2. IS 875 part 3 18

K_3 = Topography Factor

A_n = Vertical projected area of bridge m^2.

C_D, C_L, C_M = Static aerodynamic coefficient in wind axes.(To be obtained from wind tunnel test)

Calculation of design wind speed.

Basic wind speed depending upon terrain category and height of structure above ground level for 50 years return period	$\mathbf{V_b}$ =	55	m/s
Risk co-efficient based on mean permissible dsign life of 100 years and basic wind speed as per as per cl.5.3.1 table 1 of IS:875 part 3	$\mathbf{K_1}$ =	1.08	
Terrain,height and structure size factor for terain category 2 and heght of the structure above ground level as per cl.5.3.2 table 2 of IS:875 part 3	$\mathbf{K_2}$ =	1.12	
Topography factor as per cl. 5.3.3 of IS:875 part 3	$\mathbf{K_3}$ =	1	
Design wind speed cl. 5.3 of IS:875 part 3	$\mathbf{V_z} = v_b{*}k_1{*}k_2{*}k_3$ =	66.528	m/s
	$V_r = V_z \cos\alpha_0$ =	66.2748	m/s

Wind forces acting on bridge deck along wind axes.

mean drag force	$D(\alpha)$	=	$1/2{*}(\rho{*}V_z^2{*}A_n{*}C_D(\alpha))$
mean lift force	$L(\alpha)$	=	$1/2{*}(\rho{*}V_z^2{*}B{*}C_L(\alpha))$
pitching moment	$M(\alpha)$	=	$1/2{*}(\rho{*}V_z^2{*}B^2{*}C_M(\alpha))$

where,

width of deck	B =	17	m
vertical projected area	A_n =	2	m^2
air density	ρ =	1.2	kg/m^3

static aerodynamic coefficients

C_D =	0.9
C_L =	0.35
C_M =	0.2

The wind forces acting in global (undeformed) bridge axes.

$F_x(\alpha) = 1/2*(\rho*V_r^2*A_n*C_x(\alpha))$

$F_y(\alpha) = 1/2*(\rho*V_r^2*B*C_y(\alpha))$

$M_z(\alpha) = 1/2*(\rho*V_r^2*B^2*C_z(\alpha))$

where,

$C_x(\alpha) = [C_D(\alpha) - (C_L(\alpha)*B*\tan\alpha_0 / A_n)]*\sec\alpha_0$	=	0.64216
$C_y(\alpha) = [C_L(\alpha) + (C_D(\alpha)*A_n*\tan\alpha_0 / B)]*\sec\alpha_0$	=	0.36064
$C_z(\alpha) = [\ C_M(\alpha)\ ] * \sec^2\alpha_0$	=	0.20153

and

where,

α_0 =	Angle of incidence of wind	=	5	degrees
θ =	Torsional displacement of deck	=	0	degrees
α =	Eff. wind angle of attack($=\alpha_0+\theta$)	=	5	degrees

Therefore the final forces are

$F_x(\alpha)$	=	3.38474	kn/m
$F_y(\alpha)$	=	16.1572	kn/m
$M_z(\alpha)$	=	153.493	knm/m

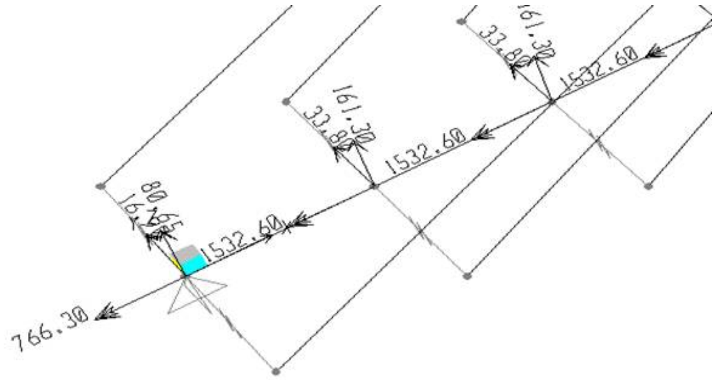


Figure 5.2: Wind loadings on deck in +y direction

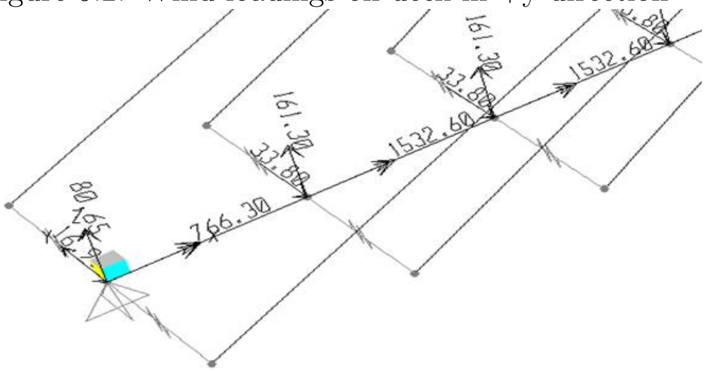


Figure 5.3: Wind loadings on deck in -y direction

5.3 Dynamic Analysis of Cable-Stayed Bridges

5.3.1 General

Cable-stayed bridges are increasing in number and popularity. This, in addition to the increase in span lengths of these flexible structures raises many concerns about their behavior under environmental dynamic loads such as wind, earthquake and vehicular traffic loads. This introduces some serious problems to the designers, since very little information are available in engineering literature concerning the response of cable stayed bridge to seismic or other dynamic loadings.

From the analysis of various observational data including the ambient forced vibration tests of cable-stayed bridges, it is known that these bridges have very small mechanical or structural damping (0.3% - 2%). These bridges occasionally experience extreme loads, especially during a strong earthquake or in a high wind environment. For such circumstances, the response should be controlled within certain limits of serviceability (human comfort) and for safety (risk of damage failure).

5.3.2 Factors Affecting the Dynamic Response

1. Three dimensionality

Because of the hybrid structural system and the flexible and extended inplane configuration as well as the three-dimensionality of the cable-stayed bridges, earthquake excitation especially to non-uniform motions, may introduce special features in to the bridge response due to the complicated interaction between the three-dimensional input motion and the whole structure.

2. Multiple-support seismic excitation

This effect is more important especially for more rigid cable-stayed bridges, for example when the deck is of reinforced or prestressed concrete or when the closely spaced multi cable system is adopted.

3. Soil condition and wave propagation.

Depending on the dynamic properties of the local soils at the supporting points, as well as the soils at the surrounding bridge site, the travelling seismic wave effect, in terms of time delay phase difference, do affects these bridges

4. Structural deformation and connections

Due to the large displacements and member forces induced due to the strong ground shaking in this type of structure, energy absorption devices and special bearings should be provided at the supporting points to dissipate seismic energy, thus assuring the serviceability of the bridge.

The response of a cable-stayed bridge to the applied loads is highly dependent on the manner in which the bridge deck is connected to the towers. If the bridge deck is swinging freely at the towers, the induced seismic forces will be kept to the minimum values but the bridge may be very flexible under service conditions (i.e., dead loads and live loads). On the other hand, a rigid connection between the deck and the towers will result in reduces movements under service conditions will attract much higher forces during an earthquake. Therefore it is recommended to provide special bearings or devices at the deck-tower connections to absorb the large seismic energy and reduce the response amplitudes.

5. Material and Geometric Non-linearity Although for the present range of effective center spans linear dynamic analysis is adequate, non-linear static analysis under dead loads is still essential to start the linear dynamic analysis from the dead load deformed state of the bridge. For the recent and future trend of longer center spans (>500m) geometrically non linear dynamic analysis as well as local material non-linear analysis (of the damping augmentation devices) are necessary for computing the response of the bridge when subjected to strong ground shaking.

6. Structural Control mechanisms Structural control basically involves the regulating of pertinent structural characteristics so as to ensure the response of the structure becomes desirable under applied loads. It results in energy dissipation from or energy input to the structure through external mechanisms the control force can be exerted by passive and or active control mechanisms. Passive control devices, which operate without using any external energy supply, are energy dissipation mechanisms activated by the motion of the structure. Active control devices operate using the external energy supply (such as hydraulic actuators) so as to counteract the effect of applied loads by providing a predictable reduction in the response.

7. Concrete and Steel design alternates For each consideration of a new cable-stayed bridge, there are two design alternates; one is mainly steel design and the other is mainly concrete design. It has been emphasized from various studies that the concrete design alternate with the rigid connection attracts higher seismic induced forces than those attracted by the steel design alternate with the loose or floating connection. Generally there is multi-modal contribution from the several modes of vibration to the total response of the structure, for both displacements and member forces.

5.3.3 Methods for Dynamic analysis

The procedures to estimate earthquake loads on cable-stayed bridges can be divided in to two main categories:
1. Pseudo- dynamic or static approach.
This can be used to roughly estimate the order of magnitude of stresses and displacements induced by earthquake loads. This approach can be further divided in to the following simplifies methods.

a. The equivalent static seismic method or seismic coefficient method.

b. The static effect of the support settlements or movements to accommodate the non-synchronous motions of the bases of the bridge.

c. The multimode spectral method, which is a pseudo-dynamic mode by mode approach.

d. The concept of the response spectrum applied in each of the three orthogonal directions separately and combined probabilistically.

2. The dynamic (time-history) response analysis. This is the refinement of the first category and is the more realistic and elaborates approach. The analysis can incorporate three-dimensionality, multiple support seismic excitations, non-linearity, soil-structure interaction, damping augmentation devices and the propagation of waves from one support to the other; such an approach is necessary and provides a more realistic way of assessing the bridge seismic response.

5.3.4 Seismic loads for Cable-Stayed Bridges

In general, cable stayed bridges are regarded as structures on which earth tremors have little effect. What distinguishes them from other structures is the fact that they rest on a limited number of point supports (abutments, pylons, piers) which can absorb different displacements during seismic action. In fact, the geological and geographical structure of the terrain can change from one pier to the next, the distance between the supports is fairly large and the speed of propagation depends on the type of wave affecting the structure. Differential movements of the supports are thus the causes of the most serious damages suffered by bridges, in particular if the seismic excitation acts along a longitudinal or transverse axis. Such damage, less marked in the case of vertical excitation, generally occurs at the junctions between the deck and the piers. In addition the bridge superstructure and towers should be designed to ensure that the response remains within the elastic range of material behavior.

Dynamic Loading

Here a linear dynamic time-history analysis is done.The time history function of El-Centro ground motion has been taken. This graph in available in SAP 200 software in time history folder which can be used as follow: → Define→ Function→ Time History

Shows the dialogue box as shown in the Fig. 5.4

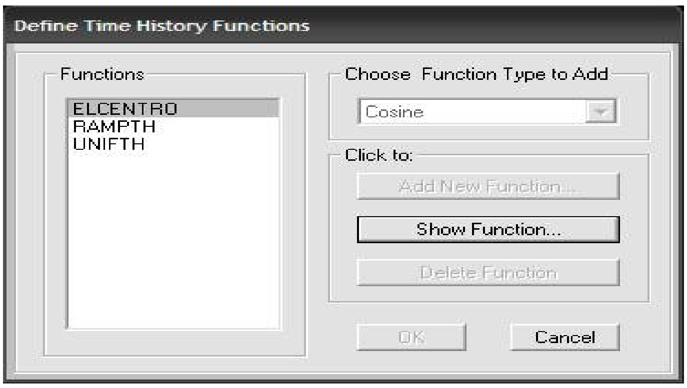


Figure 5.4: Definition for Time History Function

On further clicking add new function button open another dialogue box as shown below in the Fig. 5.5

The El-Centro ground motion graph is inserted by browsing the file from the location.

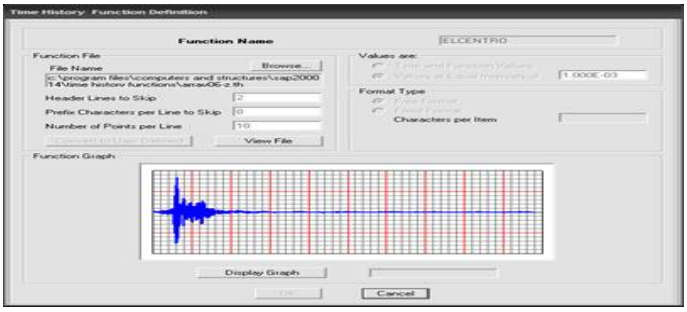


Figure 5.5: Time History Function definition dialogue box

The Time History case is also added in the analysis case on the clicking modification button following dialogue box appears and assigning load case type of Time History as shown in the Fig. 5.6

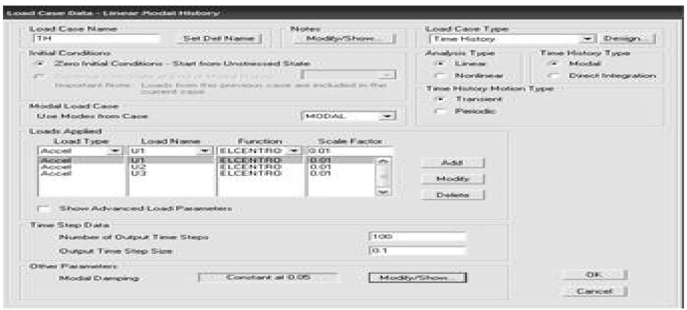


Figure 5.6: Load case Time History definition dialogue box

5.4 Analysis cases and combinations

The following analysis cases and combinations are used in the work. **Analysis cases**

• Dead

→ Consists of self weight + Superimposed Dead load + Pretension.

• M1

→ Consists of class70R vehicle applied in lanes 1 and 2 + classA in lane 3

• M2

→ Consists of class70R vehicle applied in lanes 1 and 2 + classA in lane 4

• M3

→ Consists of class70R vehicle applied in lanes 1 and 4 + classA in lane 2

• M4

→ Consists of class70R vehicle applied in lanes 1 and 4 + classA in lane 3

• M5

→ Consists of classA vehicle applied in all four lanes

• Wind+ve

→ Consists of Drag, Lift and Pitching moment applied from +ve Y-axes in the direction perpendicular to the Bridge axes.

• Wind-ve

→ Consists of Drag, Lift and Pitching moment applied from -ve Y-axes in the direction perpendicular to the Bridge axes.

• TH

→ Consists of linear dynamic analysis in the form of El-Centro ground motion.

Combinations

Linear Load Case

→ Dead + M1

→ Dead + M2

→ Dead + M3

→ Dead + M4

→ Dead + M5

→ Dead + $Wind_{+ve}$

→ Dead + $Wind_{-ve}$

→ Dead + M1 + $Wind_{+ve}$

→ Dead + M2 + $Wind_{+ve}$

→ Dead + M3 + $Wind_{+ve}$

→ Dead + M4 + $Wind_{+ve}$

→ Dead + M5 + $Wind_{+ve}$

→ Dead + M1 + $Wind_{-ve}$

→ Dead + M2 + $Wind_{-ve}$

→ Dead + M3 + $Wind_{-ve}$

→ Dead + M4 + $Wind_{-ve}$

→ Dead + M5 + $Wind_{-ve}$

Dynamic Load Case

→ Dead + M1

→ Dead + M2

→ Dead + M3

→ Dead + M4

→ Dead + M5

→ Dead + TH

→ Dead + TH

→ Dead + M1 + TH

→ Dead + M2 + TH

→ Dead + M3 + TH

→ Dead + M4 + TH

→ Dead + M5 + TH

→ Dead + M1 + TH

→ Dead + M2 + TH

→ Dead + M3 + TH

→ Dead + M4 + TH

→ Dead + M5 + TH

Results for H Pylon 200m span

The Results for H shaped Pylon of 200 m total span of the cable stayed bridge has been discussed as shown below. The various response quantities like Axial Force, Shear Force, Bending Moment, Torsion for both Linear case and Time History case has shown below.

Results for Linear case analysis

		P (k N)	V2 (k N)	V3 (k N)	T (k N m)	M2 (k N m)	M3 (k N m)
CABLES	MAX	4093.38					
	MIN	-177.76					
GIRDERS	MAX	683.51	4446.69	209.47	8583.33	8939.04	24849.38
	MIN	-26924.5	-4472.97	-209.47	-8583.33	-8939.04	-26204.3
PYLONS	MAX	1141.49	2141.77	3900.35	1196.93	20715.23	23458.69
	MIN	-32897.3	-2145.41	-3900.35	-1196.93	-23458.7	-18954.1
CROSS TOP	MAX	22.49	1068.58	50.016	10506.99	455.73	1017.99
	MIN	-2341.63	-1068.58	-50.016	-10506.99	-447.89	-7017.92
CROSS BOTTOM	MAX	8.01	1586.05	19.649	10263.66	245.18	5323.73
	MIN	-1583.85	-1586.05	-19.649	-10263.66	-240.16	-5099.53

Results for Linear case analysis

		P (k N)	V2 (k N)	V3 (k N)	T (k N m)	M2 (k N m)	M3 (k N m)
CABLES	MAX	4362.742					
	MIN	-439.754					
GIRDERS	MAX	13206.35	5648.316	13855.155	12183.08	262145.2	37089.01
	MIN	-34938.7	-5583.23	-13856.16	-12183.3	-226853	-32995.3
PYLONS	MAX	2088.894	2850.485	4889.588	2475.51	25151.76	27335.69
	MIN	-34502.2	-2784.42	-4817.527	-2527.38	-27335.7	-30077.2
CROSS TOP	MAX	69.987	2000.244	78.916	10605.26	690.7065	8513.664
	MIN	-2406.1	-2023.86	-78.917	-10605.3	-1003.24	-14740.2
CROSS BOTTOM	MAX	-1652.4	-163.373	-23.526	-5790.62	-455.766	-10592.1
	MIN	-1652.4	-138.37	-23.526	-5790.62	-468.577	-11562.4

Chapter 6

Parametric Studies

6.1 General

The behavior of cable stayed bridge is largely affected by various parameters. Some of the important parameters are:

i) Geometrical configuration of cable arrangement,

ii) Effect of height of Pylon

iii) No. of Cables

iv) Side span to central span ratio

v) Shape of Pylons

Parametric studies are carried out for four spans are selected viz. 200m, 400m, 600m and 800m (Total-span). For each span four bridge models having different pylon configuration i.e. H-Frame, A-Frame, Diamond Pylon, Delta Pylon, Inverted Y type are generated and analyzed for the loading as mentioned in previous sections.

The maximum response quantities such as Bending-moment, Shear-force, Torsion and Axial force are compared for all the four spans to find out best suitable pylon-configuration for a given conditions.

6.2 Structural Data and Parameters

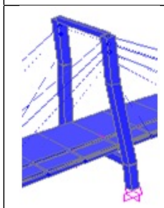
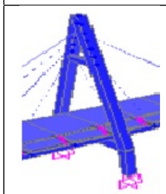
The following are the Data considered for the parametric study of the Cable Stayed Bridge which are sown in the Table 6.1.

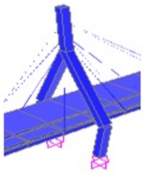
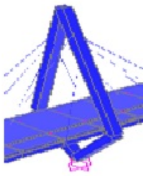
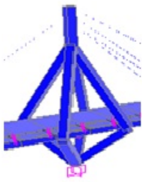
Table 6.1: Data Table

Total span m	Central span m	Side span m	Deck Dim $B \times D$ (m)	Pylon Dim $B \times D$ (m)	Pylon Ht m	Concrete Grade MPa	Steel Grade MPa
200	100	50	17×2	2×2	30	45	500
400	200	100	17×2	2×2	60	45	500
600	300	150	17×2	2.5×2.5	90	45	500
800	400	200	17×3	2.5×2.5	120	45	500

The following are the Shapes of Pylons considered for the parametric study of the Cable Stayed Bridge as shown in the table 6.2.

Table 6.2: Shapes of Pylons for study

SHAPES	**Name**	**TOTAL SPAN(m)**			
	**H type**	200	400	600	800
	**A type**	200	400	600	800

SHAPES	Name	TOTAL SPAN(m)			
	**Inverted Y**	200	400	600	800
	**Diamond**	200	400	600	800
	**Delta type**	200	400	600	800

6.3 Result of Parametric Study

The various response quantities are determined for both the Linear case and the Dynamic case for Cables, Girders and Pylons and are represented in graphical form. The various components here taken are the cable, girders and pylon which are the major structural component of the cable stayed bridge. The study is carried out in each of these components individually for both Linear cases and the Dynamic cases. As the span is more than 150m so dynamic approach is done as states in *IRC 6:2000 cl 222.5*.

6.3.1 Cable Forces

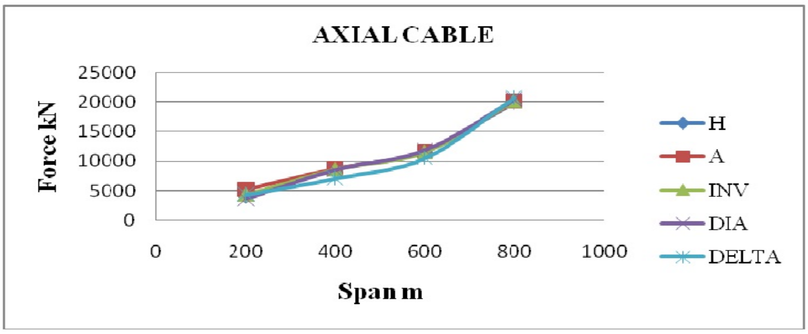


Figure 6.1: Linear case of cables

Table 6.3: Linear case of cables

Span/Shape	H	A	Inverted Y	Diamond	Delta
200	4093.38	5183.41	4316.83	3555.30	4315.59
400	8683.86	8712.45	8622.39	8502.31	7089.44
600	11689.31	11642.04	11548.71	11852.67	10515.01
800	20024	20147.11	20215.35	20493.41	20772.46

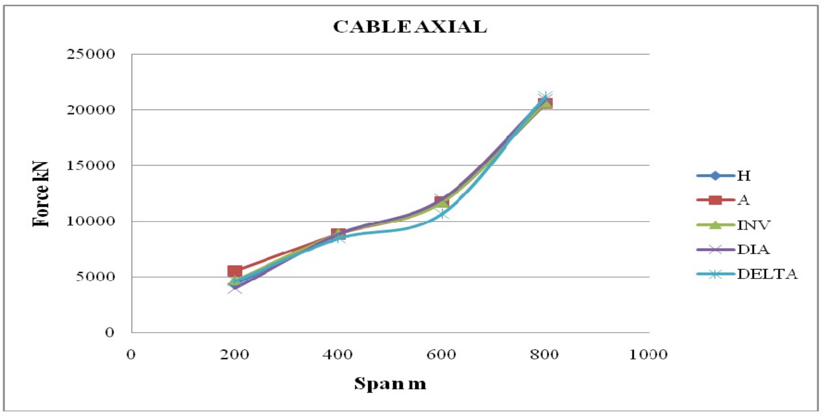


Figure 6.2: Dynamic case of cables

Table 6.4: Dynamic case of cables

Span/Shape	H	A	Inverted Y	Diamond	Delta
200	4362.74	5549.1	4707.56	3962.33	4635.03
400	8888.17	8877.58	8898.55	8828.54	8479.08
600	11795.15	11749.6	11727.75	11986.85	10680.68
800	20519.77	20623	20679.72	20981.76	21225.01

6.3.2 Girder Forces

Girder Axial Force

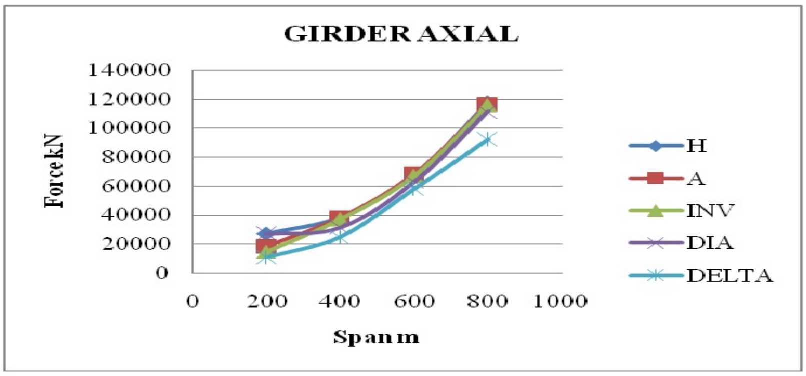


Figure 6.3: Linear case of Girder Axial

Table 6.5: Linear case of Girder Axial

Span/ Shape	H	A	Inverted Y	Diamond	Delta
200	26924.47	18537.58	14476.91	26924.47	10599.55
400	38242.38	38437.74	36924.95	31423.63	24723.5
600	67583.41	68285.83	66912.51	62818.24	57850.54
800	117658.9	116090	115597.2	111564	92112.77

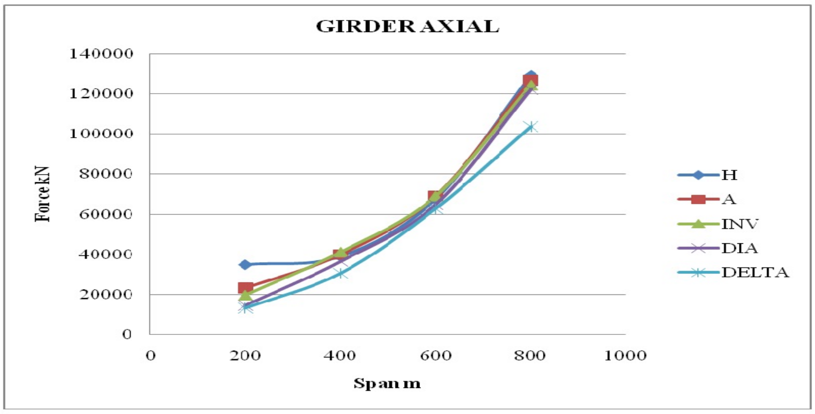


Figure 6.4: Dynamic case of Girder Axial

Table 6.6: Dynamic case of Girder Axial

Span/ Shape	H	A	Inverted Y	Diamond	Delta
200	34938.71	23335.66	19829.69	14729.11	13241.77
400	39010.22	40074.92	41218.73	36638.72	30628.11
600	67708.43	69256.66	69071.98	65014.62	62818.87
800	129250.8	126859	124255.3	122410.9	103635.4

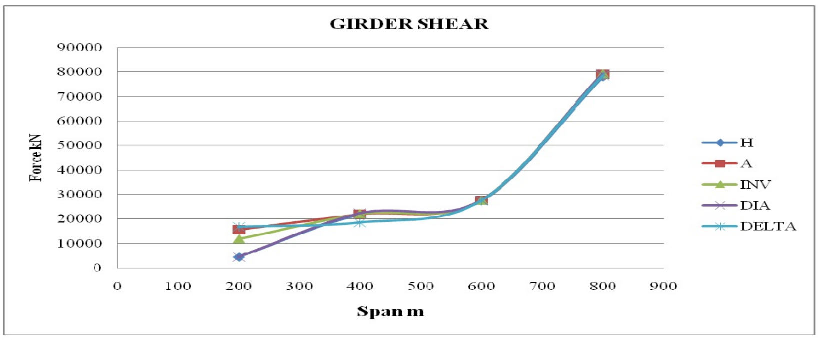


Figure 6.5: Linear case of Girder Shear

Table 6.7: Linear case of Girder Shear

Span/ Shape	H	A	Inverted Y	Diamond	Delta
200	4472.97	15667.5	11874.6	4472.97	16796.62
400	21886.35	21965.94	22155.44	22349.51	18550.02
600	27411.59	27686.26	27622.49	27618.9	27469.23
800	78103.31	78982.6	79269.48	79256.17	78603.28

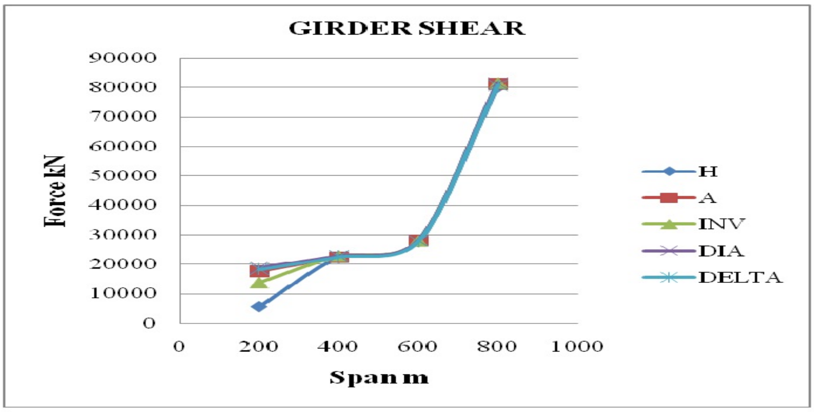


Figure 6.6: Dynamic case of Girder Shear

Table 6.8: Dynamic case of Girder Shear

Span/ Shape	H	A	Inverted Y	Diamond	Delta
200	5648.31	17523.81	13652.77	18853	18182.95
400	22231.88	22328.84	22636.54	22874.32	22548.76
600	27640.81	27959.16	27825.5	27986.89	27761.87
800	80146.51	81255.17	81407.08	81593.23	80816.59

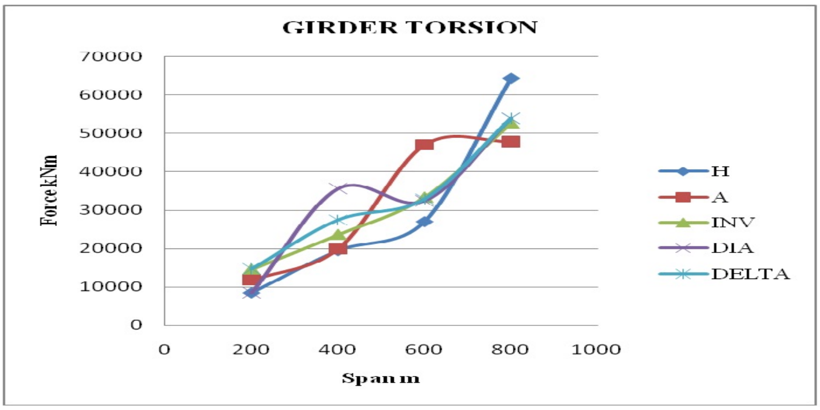


Figure 6.7: Linear case of Girder Torsion

Table 6.9: Linear case of Girder Torsion

Span/ Shape	H	A	Inverted Y	Diamond	Delta
200	8583.34	11817.62	14630.26	8583.34	14749.44
400	19561.17	20033.05	23663.46	35600.54	27508.74
600	27079.43	47049.22	33412.82	32449.57	32974.08
800	64284.91	47983.62	52667.77	53871.64	53961.77

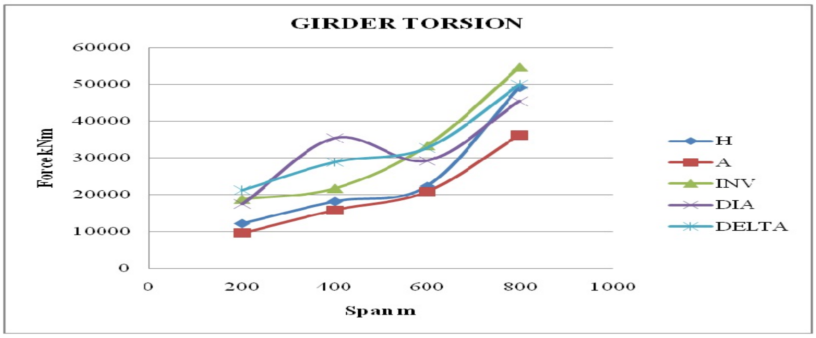


Figure 6.8: Dynamic case of Girder Torsion

Table 6.10: Dynamic case of Girder Torsion

Span/ Shape	H	A	Inverted Y	Diamond	Delta
200	12183.08	9632.002	18734.68	17483.3	21264.86
400	18151.17	15844.51	21697.35	35358.49	28915.94
600	22351.78	20847.5	33330.71	29258.36	32773.62
800	49205.72	36247.46	54630.6	45410.06	49931.38

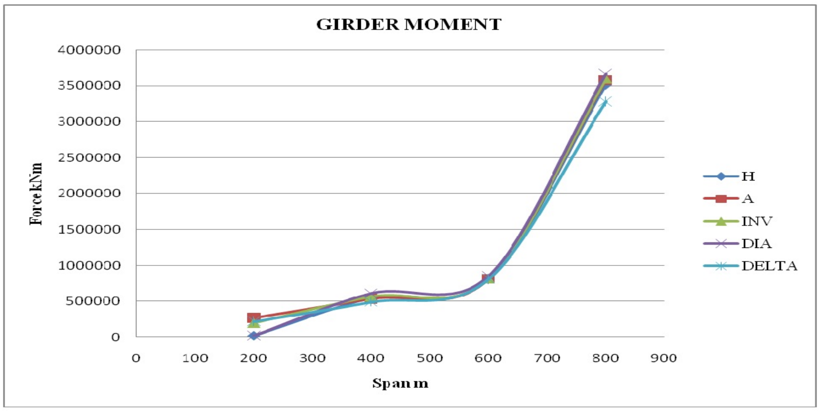


Figure 6.9: Linear case of Girder Moment

Table 6.11: Linear case of Girder Moment

Span/ Shape	H	A	Inverted Y	Diamond	Delta
200	26204.3	270236.6	197630.5	26204.33	221393
400	550663	536432.9	565387.7	614621.2	492194.6
600	813351.5	813726.9	816401.4	857264.7	812916.5
800	3516055	3579847	3599493	3673214	3279969

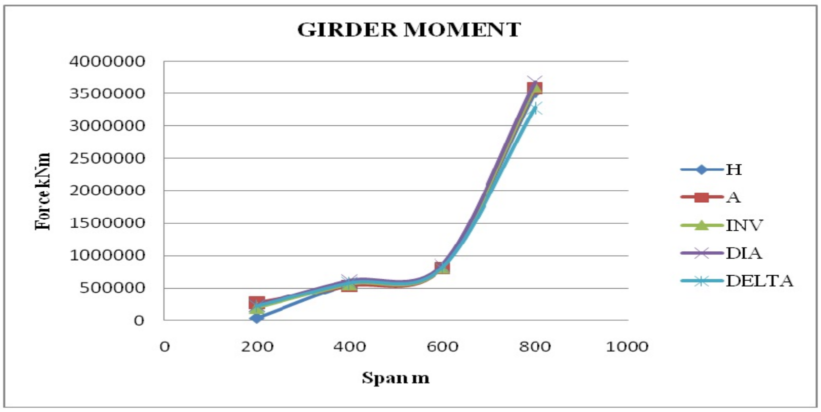


Figure 6.10: Dynamic case of Girder Moment

Table 6.12: Dynamic case of Girder Moment

Span/ Shape	H	A	Inverted Y	Diamond	Delta
200	37089.01	274531.9	201850.4	234062.8	224937
400	551033.4	537116.5	566767.9	615932	581243
600	813934.82	815244.9	818688.2	859290.6	815471.7
800	3522842.1	3584673	3604330	3678717	3284560

6.3.3 Pylon Forces

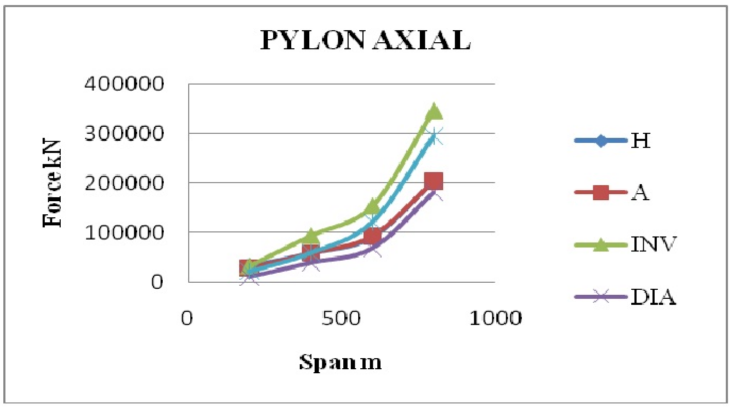


Figure 6.11: Linear case of Pylon Axial

Table 6.13: Linear case of Pylon Axial

Span/ Shape	H	A	Inverted Y	Diamond	Delta
200	32897.28	27805.28	33559.78	10973.53	22598.23
400	58693.95	58459.66	94550.42	40215.28	60335.5
600	91310.51	92991.35	154753.2	69449.97	122862.7
800	204672.8	203485.2	345961.5	182387.1	295606.8

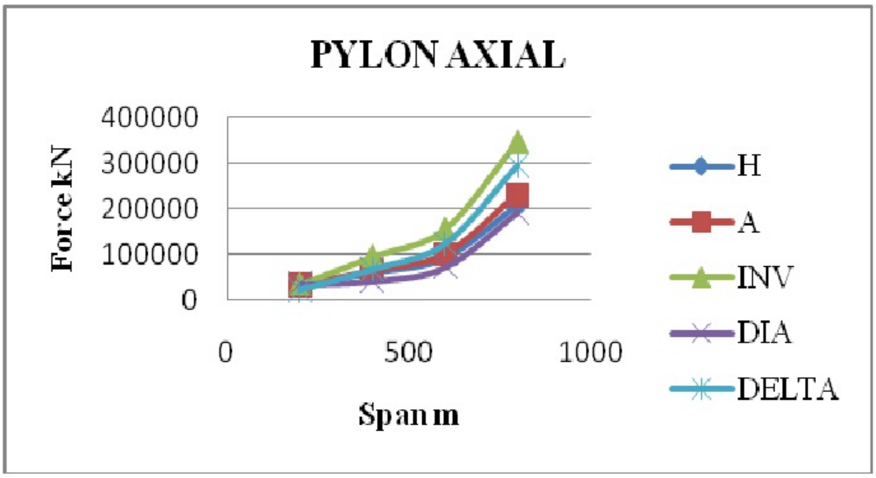


Figure 6.12: Dynamic case of Pylon Axial

Table 6.14: Dynamic case of Pylon Axial

Span/ Shape	H	A	Inverted Y	Diamond	Delta
200	34502.22	32692.29	33917.66	34502.22	22673.12
400	61013.01	63960.27	94623.79	41970.07	67128.72
600	92503.83	100516.6	155176.6	71689.99	123125.5
800	208250.8	231947.3	346301.4	191790.1	295886.2

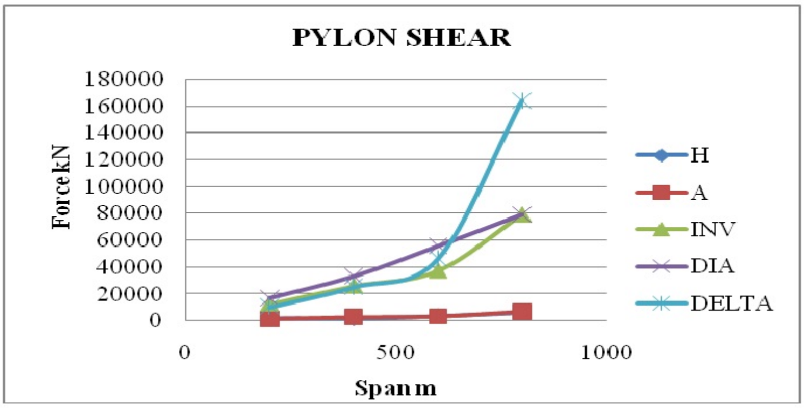


Figure 6.13: Linear case of Pylon Shear

Table 6.15: Linear case of Pylon Shear

Span/ Shape	H	A	Inverted Y	Diamond	Delta
200	2145.41	1151.06	12585.54	16971.14	10177.95
400	1711.42	2415.74	25995.03	33214.46	25098.89
600	3622.98	3084.51	37354.53	55741.53	46291.94
800	6326.38	6583.62	79269.48	79256.17	163927

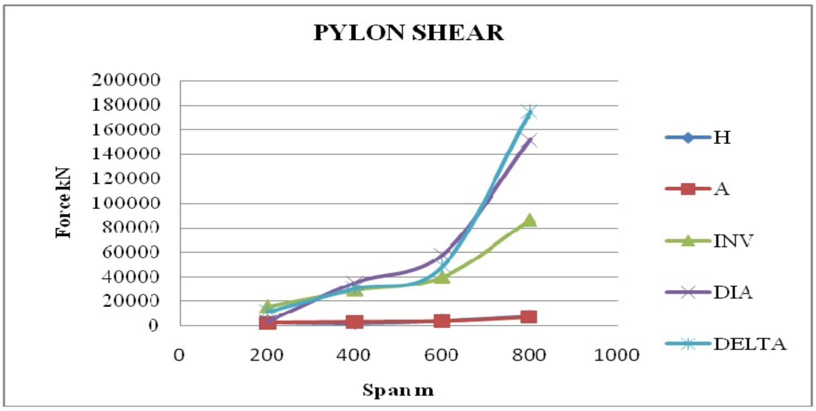


Figure 6.14: Dynamic case of Pylon Shear

Table 6.16: Dynamic case of Pylon Shear

Span/ Shape	H	A	Inverted Y	Diamond	Delta
200	2850.48	2217.18	15849.48	2850.48	11375.87
400	2147.98	2974.14	29193.17	34814.02	30440.92
600	3877.63	3533.91	40002.71	57679.06	48551.33
800	7515.41	7076.44	86712.61	152032.1	174700.6

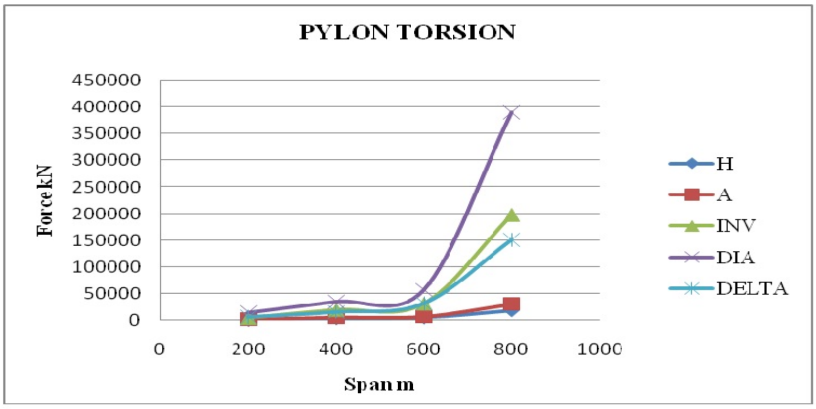


Figure 6.15: Linear case of Pylon Torsion

Table 6.17: Linear case of Pylon Torsion

Span/ Shape	H	A	Inverted Y	Diamond	Delta
200	1196.93	2123.55	4945.06	15028.38	6356.61
400	4124.95	5956.87	19790.67	35554.58	15472.99
600	5556.58	7456.22	31188.01	57480.77	31595.65
800	18788	30652.11	197633.1	390100.8	150798.8

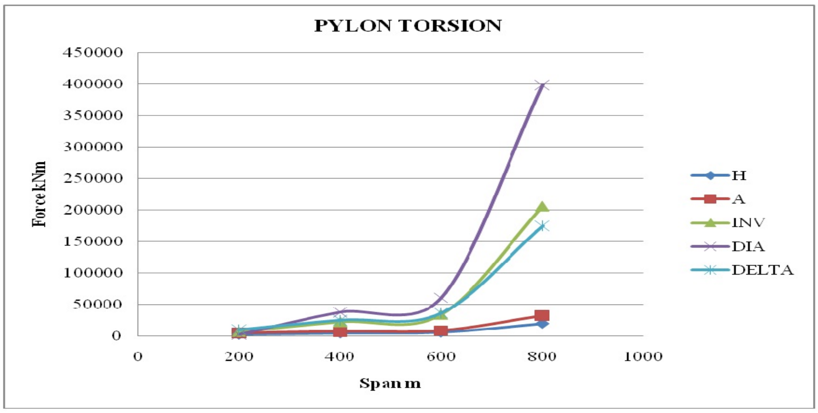


Figure 6.16: Dynamic case of Pylon Torsion

Table 6.18: Dynamic case of Pylon Torsion

Span/ Shape	H	A	Inverted Y	Diamond	Delta
200	2527.37	4660.8	6584.35	2527.37	9559.95
400	4526.49	7225.08	22223.15	37935.95	24870.89
600	5807.80	8472.59	34054.75	60683.48	36158.19
800	19717.13	32177.76	205691.2	398991.3	175194.6

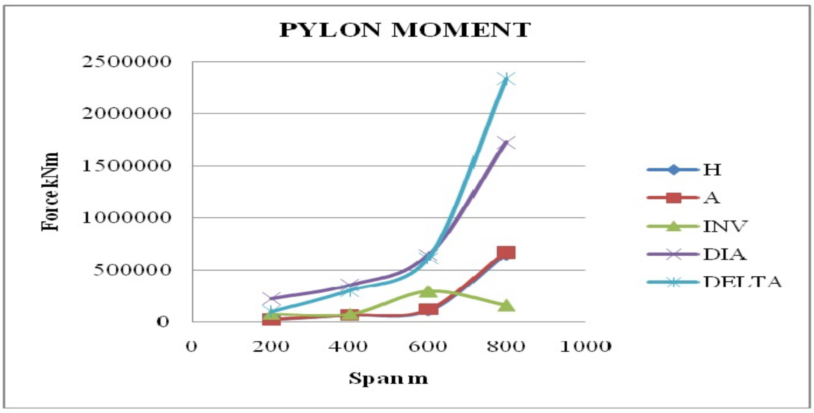


Figure 6.17: Linear case of Pylon Moment

Table 6.19: Linear case of Pylon Moment

Span/ Shape	H	A	Inverted Y	Diamond	Delta
200	23458.69	27276.5	77738.58	226615.6	100401.1
400	67783.62	71234.72	78760.57	356828.8	303159.9
600	110963.1	122982	295214	644710.6	610462.8
800	651504	673364.3	164659.1	1729107	2338349

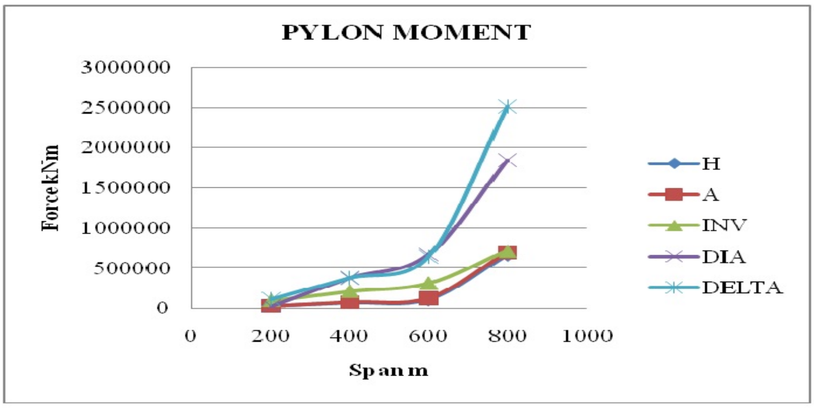


Figure 6.18: Dynamic case of Pylon Moment

Table 6.20: Dynamic case of Pylon Moment

Span/ Shape	H	A	Inverted Y	Diamond	Delta
200	30077.23	34346.97	102127.8	30077.23	117336.9
400	72318.51	84221.82	216663.6	379158.8	378678.9
600	114811.29	133966.6	315494.8	673524.4	644226.1
800	668637.31	696383.4	715638.9	1851248	2514203

6.4 Summary

- The trend line patterns of Cable in axial force in both Linear and Dynamic case for all the shapes of the pylons to the span of the cable stayed bridge remains similar as in Fig 6.1 and 6.2.

- As the span increases the axial tensile force in the cable increases. This is because the segment length and depth of the deck increases and so the number of cables also increases as in Table 6.3 and 6.4.

- In Girder Axial force the trend line pattern remains same but in Dynamic case the percentage increase in force is more than that of Linear Case as in Fig 6.3 and 6.4.

- The Delta shape of pylon has lesser force than others in both Linear and Dynamic case as in Table 6.5 and 6.6 so its performance for Girder axial is better.

- In Girder shear the Diamond shape at lesser span performs better in both but as span increases its performance also deteriorates case as in Table 6.7 and 6.8 and the trend line pattern also remains same as in Fig 6.5 and 6.6.

- In Girder Torsion the A shape Pylon performance is better with the increase in span as in Table 6.9 and 6.10 but the trend line do not have same patter in both case as in Fig.6.7 and 6.8.

- In Girder moment the trend line for both cases remains similar as in Fig.6.9 and 6.10 and the performance of all shapes also similar as in Table 6.11 and 6.12.

- In pylon Axial force the Diamond shape Performance has been better with increase in span than with other shapes as in Table 6.13 and 6.14 while the trend pattern of the graph are similar as in Fig.6.11 and 6.12.

- Pylon shear the performance of both the H type and A type pylons are far better than rest of all in both cases as in Table 6.15 and 6.16 and also from Fig.6.13 and 6.14 the pattern remains similar.

- Pylon Torsion the performance of both the A type and H type again is far better than the rest of the shapes which can been concluded form Fig. 6.15 and 6.16 and also from Table 6.17 and 6.18.

- Pylon moment the A shape,H shape and also the Inverted y shape performance is good compared with the rest two in both cases of analysis as in Table 6.19 and 6.20 and the trend line remains almost similar in both cases as in Fig 6.17 and 6.18.

- Thus form all the above discussion the performance of different shape are better for different response quantities so it cannot be merely concluded form this analysis which is better so a detailed cost comparison can only give the better shape suitable.

Chapter 7

Design of Cable Stayed Bridge

7.1 General

7.2 Design of Superstructure

7.2.1 Cables

The cables are designed for the maximum value of axial tension form various load combinations. There is symmetry in the cable arrangement so the six cable are shown here as design. Cables are assumed to be capable of taking tensile force as well as compressive force. Compressive forces that occur on account of applied live loads are usually small, if at all they occur.

It is implied that dead load tension and prestress in the cables are much larger than the compressive force and hence cables do not become slack.Cables are assumed to be straight members, that is; the effect of catenary action due to self-weight of cables is neglected.

For the Design, 7mm wires are considered with the following Properties.

Wire cross Section Area = 38.48 mm^2

Wire Ultimate Tensile Strength (UTS) = 1770 N/mm^2

Maximum Allowable Stress = 0.42 UTS

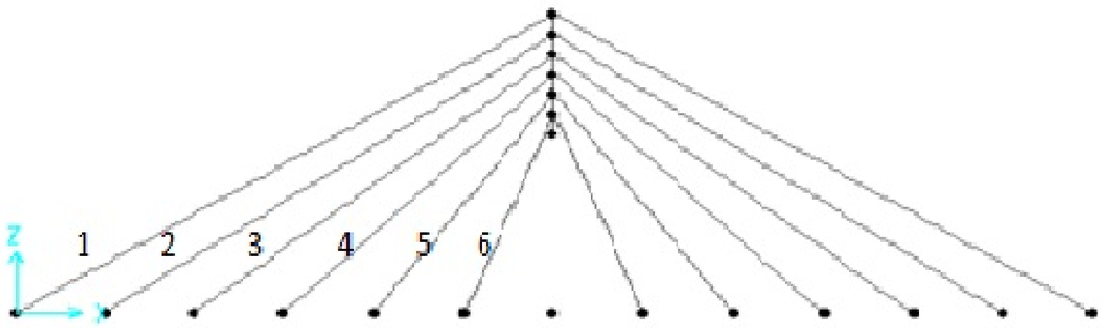


Figure 7.1: Cable Arrangement

As shown in the Fig. 7.1 the cable of only one side is designed as there is symmetry.The are of each wire is taken and it capacity is found out from the Ultimate Tensile Stress(UTS). Then the maximum axial force obtained from the analysis for various combination of loading is taken. On division of the maxium load to the capacity give the required number of wire.From the areas provided divided by the area of each wire gives number of wire required which is checked with the required number which is correct as discussed in the Table 7.1.

Table 7.1: Cable Design

	AREA mm^2	**CAPACITY kN**	**AXIAL LOAD kN**	**NO REQ**	**AREA PRV** mm^2	**NO PRV**	**CHECK**
1	38.48	69.27	4363	62.98	4000	104	**OK**
2	38.48	69.27	4065	58.68	4000	104	**OK**
3	38.48	69.27	3684	53.18	3000	78	**OK**
4	38.48	69.27	3219	46.47	3000	78	**OK**
5	38.48	69.27	2871	41.45	2000	52	**OK**
6	38.48	69.27	3587	51.78	2000	52	**OK**

7.2.2 Pylons

Pylons are designed as slender columns. The forces taken are the maximum obtained from SAP200 for all the combinations of loads.Below is the give data and design steps. The detailing of the pylon is also done as shown in Fig 7.2

B		2000	mm	
D		2000	mm	
f_{ck}		45	MPa	
f_y		500	MPa	
Axial		34502	kN	
M_{22}		25152	kNm	
M_{33}		15299	kNm	
HT		40000	mm	
L_{eff}		48000	mm	
COVER		100	mm	
SLENDERNESS				
L_{ex}/D	24	>	12	LONG CLOUMN
L_{ey}/B	24	>	12	LONG CLOUMN
THE COLUMN IS SLENDER IN BOTH DIRECTION				
ADD ECCENTRICITY		TB-1		
	L_{ey}/B	e_x/D		
	24	0.2904		
ADDITIONAL MOMENTS				
M_{ax} $\quad P_u e_x D$		20038.76	kNm	
M_{ay} $\quad P_u e_y B$		20038.76	kNm	
MAJOR AXIS				
Let				

P_t%			3.6%		
A_g			4000000	mm^2	
SP 16	**CHART**		**63**		
P_{uz}/Ag			26	N/mm^2	
P_{uz}			104000	kN	
Let	Dia	=	32	mm	
MOMENT REDUCTION					
	d'/D_{xx}		0.05		
	d'/D_{yy}		0.05		
USING	**TABLE**	**60**	**SP 16**		
k_1			0.21		
k_2			0.54		
Pb CALCULATION					
Pb_{xx}	=	(k_1+k_2p/f_{ck})f_{ck}BD			
			47268	kN	
Pb_{yy}	=	(k_1+k_2p/f_{ck})f_{ck}BD			
			47268	kN	
k_x	$P_{uz} - P_u/P_{uz} - P_{bx}$		1.22		
k_y	$P_{uz} - P_u/P_{uz} - P_{by}$		1.22		
M_{ax}			24547.94	kNm	
M_{ay}			24547.94	kNm	
MOMENT WITH MIN ECCENTRICITY					
e_x	=l/500+D/30		146.6 mm	>	20 mm e_{min}
e_y	=l/500+D/30		146.6 mm	>	20 mm e_{min}

MOMENTS DUE TO MIN ECCENTRICITY			
M_{ux}	5060.29	kNm	
M_{uy}	5060.29	kNm	
TOTAL MOMENT			
M_{ux}	29608.23	kNm	
M_{uy}	29608.23	kNm	
TOTAL MOMENT WITH EXTERNAL LOAD			
M_{ux}	44907.24	kNm	
M_{uy}	54760.24	kNm	
MAJOR AXIS			
p/f_{ck}	0.08		
CHART **47**			
P_u/f_{ck}BD	0.19		
Mu/$f_{ck}BD^2$	0.165		
M_{ux}	59400	kNm	**OK**
MINOR AXIS			
Let			
P_t%	3.6%		
P_t/f_{ck}	0.08		
d'/D	0.05		
P_u/f_{ck}BD	0.19		
CHART47 **SP 16**			
Mu/$f_{ck}BD^2$	0.165		
M_{uy}	59400	kNm	**OK**
M_{ux}/M_{ux1}	0.49		
M_{uy}/M_{uy1}	0.49		

P_u/P_{uz}	0.33			
SP 16 CHART64				
P_u/P_{uz}	0.2	>	0.33	**OK**
Area of steel	144000	mm^2		
Dia of bar		32		
Area	804.24	mm^2		
No	179.04	NOS		
Provided	180	nos		

The Cross sectional detailing of the pylon is as shown in the Fig. 7.2

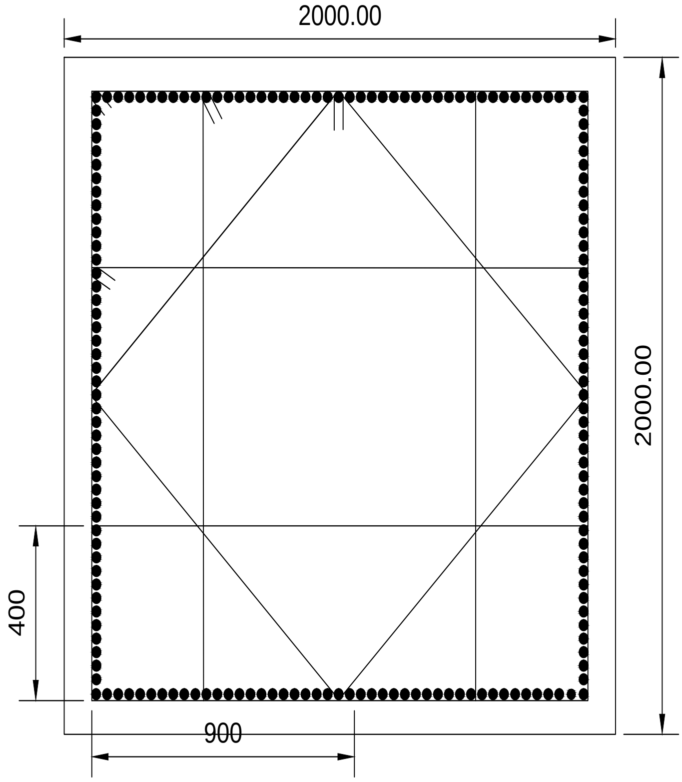


Figure 7.2: Detailing of Pylons

7.2.3 Box Girder

The sectional properties are also found from the section detailer of SAP 2000.The Max values of the Bending Moments, Shear Force, as in Table 7.2 are taken Uniform through out the segment length of Girder considered for design of 8m as this will be the worst governing case.

Section properties of girder

• Area = 18.12 m^2

• Centroid:

→ X : 8.5 m

→ Y : 0.967 m

• Moments of Inertia:

→ X : 9.46 m^4

→ Y : 394.69 m^4

• Radius of Gyration:

→ X : 0.72 m

→ Y : 4.67 m

• Section Modulus at top and bottom :

→ Z_b = I/y_b = 9.79 m^3

→ Z_t = I/y_t = 9.15 m^3

Table 7.2: Analysis Result

Dist m	**BM DL tm**	**BM SIDL tm**	**BM LL tm**	**SF DL t**	**SF SIDL t**	**SF LL t**	**TORSION tm**
0	1341	875	2621	394	65	221.20	172
0.8	1341	875	2621	394	65	221.20	172
1.6	1341	875	2621	394	65	221.20	172
2.4	1341	875	2621	394	65	221.20	172
3.2	1341	875	2621	394	65	221.20	172
4	1341	875	2621	394	65	221.20	172

General

A 6-lane box girder bridge is considered here for the study. The salient features are as follows:

- Length of Girder = 8 m
- Effective Span = 8 m
- Height of Girder = 2 m
- Overall Width = 17 m

Material and Permissible Stresses:

• Grade of Concrete	= M 45
• Grade of Steel(for reinforcing bars)	= f_y 500
• Prestressing Steel	= 15.2 mm dia 7ply class II Low relaxation strands confirming to *IS : 14268-1995.*
• Modulus of elasticity of steel (E_s)	= 1.95 x 10^5 MPa
• Area of prestressing cable	= 2660 mm^2
• Breaking Load of 19T15 Cable	= 504.92 t

Permissible Stresses in Concrete

• Permissible temporary compressive stress in concrete at 7th day i.e., at time of prestressing (cl. 7.1.3, IRC : 18-2000);

$\rightarrow$ = 0.5 f_{cj} or 200 kg/cm^2 , whichever is less.

$\rightarrow$ = 0.5 $\times$ 360

$\rightarrow$ = 180 kg/cm^2

•Permissible temporary tensile stress in concrete *(cl. 7.1.4, IRC : 18-2000)*

$\rightarrow$ = (1/10) x Permissible temporary compressive stress in concrete

$\rightarrow$ = 18 kg/cm^2

• Permissible compressive stressing concrete during service *(cl. 7.2.1, IRC : 18-2000)*

$\rightarrow$ = 0.33 f_{ck} = 0.33 $\times$ 450 = 148.5 kg/cm^2

Sections Under Consideration Sections under consideration from centreline of bearings, i.e., support, which are checked are listed below:

→	0.5 L	=	4 m	
→	0.4 L	=	3.2 m	
→	0.3 L	=	2.4 m	
→	0.2 L	=	1.6 m	
→	0.1 L	=	0.8 m	
→	0.0 L	=	0 m	where, L = 8m.

Cable Layout

In the Box girder , 33 cables of 19T15 are used and all are stressed in one stage. As per *IRC : 18-2000, cl.22*, spare cables to the extent of 4% of total prestress are provided. Hence, spare cables required = 4% of (33×19)

→ = 25 strands

i.e., two spare cables of 19T15 in the soffit are provided.

Out of 33 cables, 12 cables are anchored in the webs and remaining are anchored in the soffit at the ends of the box as shown in Fig.7.3 and 7.4

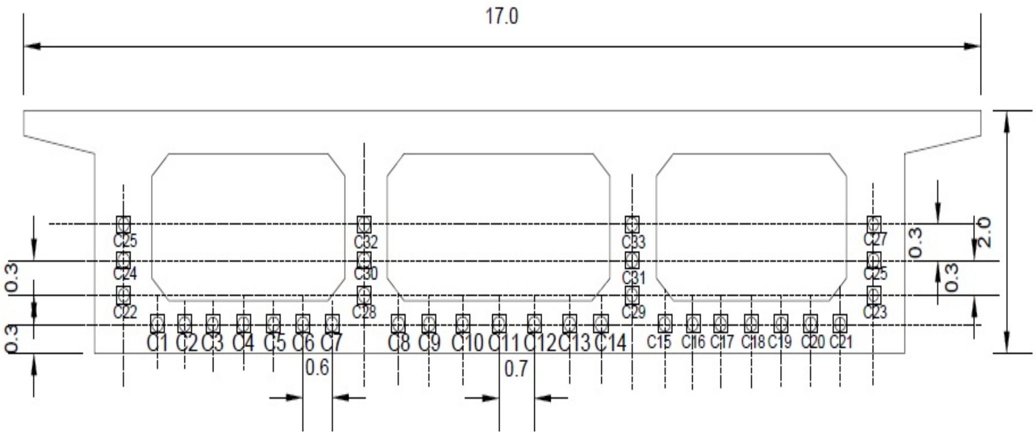


Figure 7.3: Cable layout at Support-section

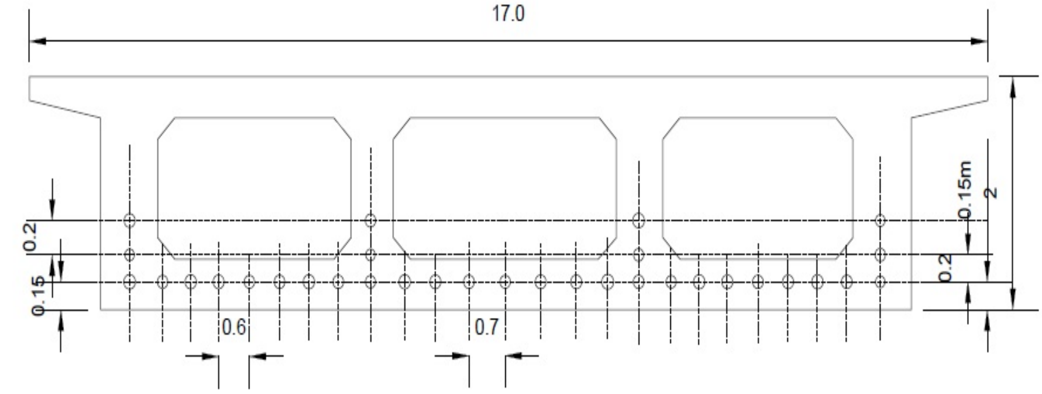


Figure 7.4: Cable layout at Mid-section

Temperature Effect

(IRC - 6:2000, cl. 218.4 and Fig. 10)

The possitive temperature effect is calculated considering the temperature variation diagram as in Fig 7.5

→	α_t	=	Co-efficient of Thermal Expansion
→		=	$1.17 \times 10^{-5}/C$ *(IRC - 18-2000, cl. 6)*
→	E_c	=	$33541020\ kN/m^2$

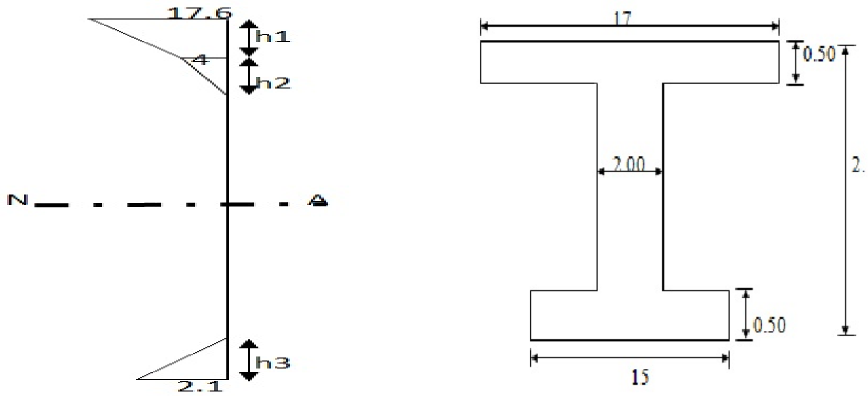


Figure 7.5: Possitive Temperature Effect

→ h1 = 0.3 h

→ = 0.3 × 2

→ = 0.6 m > 0.15 m

→ h1 = 0.15 m

→ h2 = 0.3 h

→ = 0.3 × 2

→ = 0.6 m > 0.25 m

→ h2 = 0.25 m

→ h3 = 0.3 h

→ = 0.3 × 2

→ = 0.6 m > 0.15 m

→ h3 = 0.15 m

Table 7.3: Positive Temperature Effect

No.	**Forces (kN)**	**Distance of CG from NA (m)**	**Moment (kN.m)**
1	10807.52	0.87	9470.19
2	3335.65	0.734	2447.26
3	1050.73	-0.983	-1032.87
	Total	15193.73	10884.58

•Stress at top of girder from Table 7.3,

→ = -P/A -M/Z + $\alpha_t t h_1$

→ = -(15193.73/18.12) - (10884.58/9.78) + (1.17x10$^-$5x17.6x33541020)

→ = 495.6 t/m^2

•Stress at bottom of girder,

→ = -P/A -M/Z + $\alpha_t t h_1$

→ = -(15193.73/18.12) - (10884.58/9.15) + (1.17x10-5x2.1x33541020)

→ = 117.47 t/m^2

The reverse temperature effect is calculated considering the temperature variation diagram as in Fig. 7.6

$\rightarrow$ α_t = Co-efficient of Thermal Expansion

$\rightarrow$ = $1.17 \times 10^{-5}/C$ *(IRC - 18-2000, cl. 6)*

$\rightarrow$ E_c = $33541020\ kN/m^2$

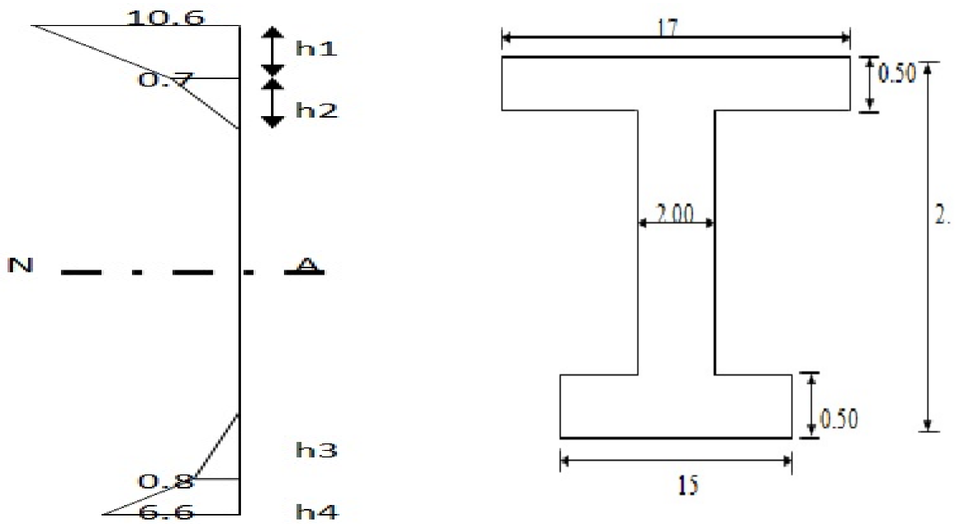


Figure 7.6: Reverse Temperature Effect

$\rightarrow$ h1 = h4 = 0.2 h

$\rightarrow$ = 0.2 × 2

$\rightarrow$ = 0.4 m > 0.25 m

$\rightarrow$ h1 = h4 = 0.15 m

$\rightarrow$ h2 = h3 = 0.25 h

$\rightarrow$ = 0.25 × 2

$\rightarrow$ = 0.5 m > 0.2 m

$\rightarrow$ h2 = h3 = 0.2 m

Table 7.4: Reverse Temperature Effect

No.	**Forces (kN) (kN)**	**Distance of CG from NA (m)**	**Moment (kNm)**
1	9423.22	0.81	7590.36
2	466.99	0.65	303.7
3	533.70	-0.84	-448.42
4	4936.78	-0.91	-4477.03
	Total	15360.7	2968.62

•Stress at top of girder from Table 7.4,

→ = P/A + M/Z - $\alpha_t h_1$

→ = (15360.7/18.12) + (2968.62/9.780) - (1.17x10-5 x 10.6 x 33541020)

→ = -300.89 t/m^2

•Stress at bottom of girder,

→ = P/A - M/Z - $\alpha_t h_1$

→ = (15360.7/18.12) + (2968.62/9.150) - (1.17x10-5 x 6.6 x 33541020)

→ = -206.65 t/m^2

Losses in Prestress

The forces in prestressing cables suffers losses due to the following factors:

a. Friction due to strands and sheathing.

b. Slip of wedges at the anchor.

c. Elastic shortening of concrete.

d. Relaxation of Prestressing force.

e. Shrinkage of concrete.

f. Creep of concrete.

Losses due to (a), (b), (c) take place immediately while stressing the cables and on transfer of force to girder and so are termed as Instantaneous Loss. Force that is available at this stage is the "Initial Presstressing Force Available". After this, losses continue to take place in the cables due to (d), (e) and (f) over a long period and referred as "Time Dependent Loss". This all losses are calculated in following sections.

Loss In Prestress Due To Friction:-

As per *IRC : 18-2000, cl. 11.6, Table - 5,*

$\rightarrow$ k $= 0.002/\text{m}$

$\rightarrow$ $= 0.17/\text{radian}$

$\rightarrow$ $P_x = P_{je}^{-(\mu\theta+kx)}$

Here there is simultaneous change of angle in vertical as well as horizontal planes, therefore effective angle is , is used, where v is the change in angle in vertical plane and h is the change in horizontal plane. Effective angles for all cables are as in Table 7.5 below:

Table 7.5: Effective Angles for Cables

Section	**L @ end m**	**Cable 1-21** $\Delta\theta$	**Cable 22-23** $\Delta\theta$	**Cable 24-25** $\Delta\theta$	**Cable 26-27** $\Delta\theta$	**Cable 28-29** $\Delta\theta$	**Cable 30-31** $\Delta\theta$	**Cable 32-33** $\Delta\theta$
1-1	0	0	0	0	0	0	0	0
2-2	0.8	0.137	0.053	0.086	0.121	0.053	0.086	0.121
3-3	1.6	0.154	0.108	0.175	0.253	0.108	0.175	0.253
4-4	2.4	0.162	0.164	0.268	0.392	0.268	0.268	0.392
5-5	3.2	0.162	0.220	0.345	0.447	0.220	0.345	0.447
6-6	4	0.162	0.246	0.345	0.447	0.246	0.345	0.447

As per *IRC : 18-200, cl. 8* maximum jacking stress P_j should be less than 90% of 0.1% of proof stress.

Here, 0.1% of proof stress = 0.85 x min. UTS

Maximum jacking stress = 0.85 x 0.90 x min. UTS

= 145.2 kg/cm^2, i.e., 76.5% of UTS.

Due to friction stress changes continuously and values are evaluated and presented in Table 7.6 below.

Table 7.6: Stress after friction loss

Section	**Dist. from end (L) m**	**Cable 1-21** kg/cm^2	**Cable 22-23** kg/cm^2	**Cable 24-25** kg/cm^2	**Cable 26-27** kg/cm^2	**Cable 28-29** kg/cm^2	**Cable 30-31** kg/cm^2	**Cable 32-33** kg/cm^2
1-1	0	145.2	145.2	145.2	145.2	145.2	145.2	145.2
2-2	0.8	141.62	143.65	142.87	142.01	143.65	142.87	142.01
3-3	1.6	140.99	142.10	140.48	138.63	142.10	140.48	138.63
4-4	2.4	140.58	140.54	138.07	135.17	140.54	138.07	135.17
5-5	3.2	140.36	138.98	136.05	133.71	138.98	136.05	133.71
6-6	4	140.13	138.13	135.83	133.49	138.13	135.83	133.49

Loss In Prestress Due To Anchorage Slip

After stressing the cable at the time of releasing the jack, there is a slip at the anchorage causing further loss in prestress. This is also called as "Reverse Friction" and this behaviour is exactly the same as that of stressing, but in the reverse direction. The amount of slip is taken as 6 mm (as recommended).

The diagram shown in Fig.7.7 represents the stress variation over the length while stressing and after slip. The area enclosed between the two curves is equivalent to slip value (Δ) multiplied by Young's Modulus of Strand (E_s) i.e., $E_s \times x$. Points 1-2-3-4-5-6 represent the prestressing stress while jacking and 1'-2'-3'-4'-5-6 represent prestressing stress after slip. The shaded area is symmetrical about the horizontal through the zero slip point, which is obtained by trial and error.

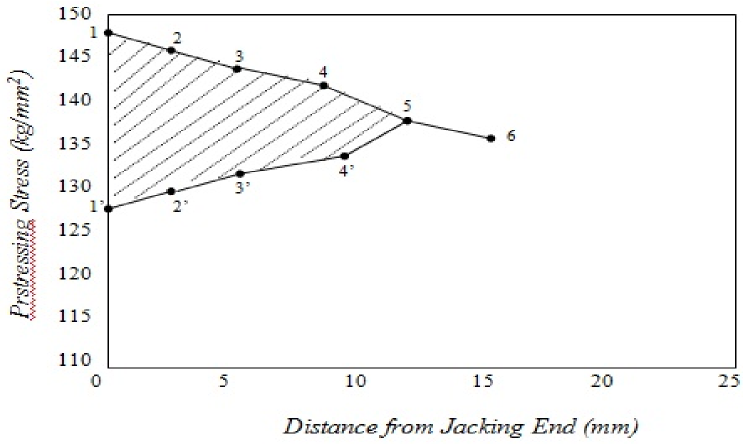


Figure 7.7: Stress variation diagram

Loss due to slip is for Cable 1-21 is calculated as below in Table 7.7 and the stress variation diagram as shown in the Fig.7.8

Table 7.7: Slip loss in cable 1-21

Dist from end L in m	**Stresses before slip** kg/mm^2	**Slip loss point m**	**Stress at slip loss point** kg/mm^2	**Stresses after slip** kg/mm^2	**Dist from end m**	**Stresses before slip** kg/mm^2	**Stresses after slip** kg/mm^2
0	145.2	2.7	140.5	135.81	0	145.2	135.81
0.8	141.62	2.7	140.5	139.38	0.8	141.62	139.38
1.6	140.99	2.7	140.5	140.01	1.6	140.99	140.01
2.4	140.58	2.7	140.5	140.42	2.4	140.59	140.42
3.2	140.36	2.7	140.5	140.50	2.7	140.50	140.50
4	140.14	2.7	140.5	140.139			

From the Fig.7.8 and Table 7.7 the average stress due to slip can be summarised as follows :

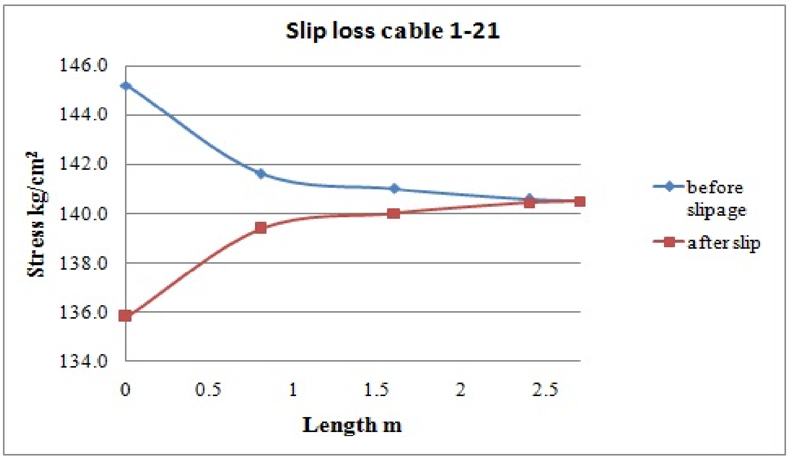


Figure 7.8: Stress variation diagram for cable 1-21

Average stress before slip = 71.44 kg/mm^2

Average stress after slip = 69.06 kg/mm^2

Area of stress diagram = 6.43 kg/mm^2

Similarly, the results for other cables are obtained and summarised in the Table 7.8.

Table 7.8: Summary of slip loss in all cables

Section	**Dist. from end (L) m**	**Cable 1-21** kg/cm^2	**Cable 22-23** kg/cm^2	**Cable 24-25** kg/cm^2	**Cable 26-27** kg/cm^2	**Cable 28-29** kg/cm^2	**Cable 30-31** kg/cm^2	**Cable 32-33** kg/cm^2
1-1	0	135.80	134.87	130.10	125.04	134.93	130.02	124.99
2-2	0.8	139.38	136.41	132.42	128.23	136.47	132.34	128.18
3-3	1.6	140.01	137.97	134.81	131.61	138.03	134.73	131.55
4-4	2.4	140.42	139.53	137.22	135.06	139.59	137.14	135.01
5-5	3.2	140.50	138.98	136.05	133.71	138.98	136.05	133.71
6-6	4	140.13	138.13	135.83	133.49	138.13	135.83	133.49

Net Prestressing Force after Friction and Slip

Net Prestressing force available is calculated at various sections considering the prestressing stress after slip loss and dividing by total area of cables. The horizontal component of prestressing force is calculated in Table 7.9 below for further considerations in calculations.

Table 7.9: Horizontal component of prestressing force(t)

CABLES	**Sec 1-1**	**Sec 2-2**	**Sec 3-3**	**Sec 4-4**	**Sec 5-5**	**Sec 6-6**
1-21	356.54	370.65	372.42	373.52	373.74	372.77
22-23	347.93	356.15	363.51	369.89	369.57	367.44
24-25	325.67	340.46	353.44	363.93	361.90	361.32
26-27	299.90	323.14	343.51	358.74	355.67	355.10
28-29	348.09	356.31	363.67	370.05	369.57	367.44
30-31	325.47	340.26	353.23	363.72	361.90	361.32
32-33	299.77	323.00	343.37	358.60	355.67	355.10
TOTAL	6485.75	6765.58	6905.37	7012.64	6998.84	6979.30

Loss Calculation due to Elastic Shortening, Relaxation of Steel and Creep and Shrinkage

Table 7.10: Sectional Properties

Sec	**Prest-ressing force ton**	**Area Girder** m^2	**MI Girder** m^4	**C.G. Girder btm** m	**C.G. cables** m	**Ecc m**	**BM DL** $t.m$	**BM SIDL** $t.m$	**BM LL** $t.m$
1-1	6485.75	18.12	9.46	1.03	0.52	0.52	1341.13	875	2621
2-2	6765.58	18.12	9.46	1.03	0.36	0.67	1341.13	875	2621
3-3	6905.37	18.12	9.46	1.03	0.27	0.76	1341.13	875	2621
4-4	7012.64	18.12	9.46	1.03	0.23	0.80	1341.13	875	2621
5-5	6998.84	18.12	9.46	1.03	0.22	0.81	1341.13	875	2621
6-6	6979.30	18.12	9.46	1.03	0.22	0.81	1341.13	875	2621

The section properties are as shown in the Table 7.10 for further calculation.

Calculation for Section 1-1 :

Total Prestressing Force	= 6485.75 ton
C.G. of Cables	= 0.51 m
C.G. of Girder	= 1.03 m
Eccentricity	= 1.03 – 0.51
	= 0.52 m

Stress at C.G. of Cables = $\frac{P}{A} + \frac{P_e^2}{I} - \frac{M_{girder}.e}{I}$

Where, M girder = bending moment due to self-weight = 1342.13 tm

A = sectional area of girder = 18.1172 m^2

I = moment of inertia of girder = 9.4591 m^4

Hence, stress at C.G. of cables = 467.01 t/m^2

Now, **Loss due to elastic shortening**

$$= \frac{467.01 \times 0.5 \times 1.95 \times 10^5 \times 140 \times 627 \times 10^{-6}}{33541.02}$$

$$= 115.55 ton$$

$\therefore$ Net prestressing force (after elastic shortening)

= 6485.75 – 115.55 = 6370.2 ton

Loss due to relaxation of steel

Average stress in cable immediately after anchoring

= $\frac{6370.2}{140 \times 627}$ = 0.0726 t/mm^2

Now, Ultimate Tensile strength (UTS) = $\frac{260700}{140 \times 9.81}$ = 189.82 kg/mm^2

$\therefore$ Ratio of average stress to UTS = $\frac{0.1255 \times 1000}{189.8208}$ = 0.38%

Loss due to Creep and Shrinkage in Concrete between 7 to 90 days:

Stress at C.G. of Cables = $\frac{P}{A} + \frac{P_e{}^2}{I} - \frac{M_{girder}e}{I}$

= 457.38 t/m^2

Shrinkage strain (*cl. 11.3 and Table-3, IRC: 18-2000*),

= (3.5 – 1.5) $\times 10^{-4}$

$= 2 \times 10^{-4}$

Creep strain (*cl. 11.4 and Table-2, IRC : 18-2000*),

$= (5.1 - 4) \times 10^{-4} \times \frac{457.38}{1000}$

$= 5.0 \times 10^{-5}$

$\therefore$ Loss due to creep and shrinkage,

$= (2 + 0.5) \times 10^{-4} \times 627 \times 140 \times 1.95 \times 10^{4}$

$= 438.46$ ton

Increasing the loss by 20 % = 1.2×438.46

$= 514.15$ ton

$\therefore$ Net prestressing force (after deducting shrinkage and creep)

$= 6370.20 - 514.15$

$= 5856.04$ ton

Loss due to Creep and Shrinkage in Concrete beyond 90 days :

Stress at C.G. of Cables $= \frac{P}{A} + \frac{P_e{}^2}{I} - \frac{M_{girder+SIDL}.e}{I}$

$= 366.9\ t/m^2$

Shrinkage strain (*cl. 11.3 and Table-3, IRC : 18-2000*),

$= (1.5 - 0)$ x 10^{-4}

$= 1.5 \times 10^{-4}$

Creep strain (*cl. 11.4 and Table-2, IRC : 18-2000*),

$= (4 - 0) \times 10^{-4} \times \frac{366.9}{1000}$

$= 1.47 \times 10^{-4}$

$\therefore$ Loss due to creep and shrinkage,

$= (1.5 + 1.47) \times 10^{-4} \times 627 \times 140 \times 1.95 \times 10^{4}$

$= 507.97$ ton

Increasing the loss by 20 % = 1.2×507.97

$= 609.56$ ton

$\therefore$ Net prestressing force (after deducting shrinkage and creep)

$= 5856.04 - 609.56 = 5246.48$ ton $\therefore$Percentage loss = 19.11 %

Similarly, the loss calculations for other sections are summarized in Table 7.12.

Table 7.11: Summary of Loss calculation

	SEC 1-1	**SEC 2-2**	**SEC 3-3**	**SEC 4-4**	**SEC 5-5**	**SEC 6-6**
Total Prestressing Force (ton)	6485.75	6765.58	6905.37	7012.64	6998.84	6979.30
C.G. of Cables (m)	0.52	0.36	0.27	0.23	0.22	0.22
C.G. of Girders (m)	1.03	1.03	1.03	1.03	1.03	1.03
Eccentricity (m)	0.52	0.67	0.76	0.80	0.81	0.81
Stress at C.G. of Cables (t/m^2)	467.01	601.06	699.01	748.86	757.08	755.30
Loss due to Elastic Shortening (ton)	115.55	148.72	172.96	185.29	204.90	192.73
Net Prestressing Force (after elastic shortening (ton))	6370.20	6616.86	6732.41	6827.34	6793.94	6786.58
Loss due to relaxation of steel						
Avg. stress in cable immediately after anchoring	0.07	0.08	0.08	0.08	0.08	0.08
(t/mm^2)						
Ultimate Tensile Strength (UTS) (kg/mm^2)	189.82	189.82	189.82	189.82	189.82	189.82
Ratio of avg. stress to UTS	0.38	0.40	0.40	0.41	0.41	0.41
Relaxation loss (%) (*IRC : 18-2000*)	0.00	0.00	0.00	0.00	0.00	0.00
Relaxation loss (ton)	0	0	0	0	0	0
Relaxation loss beyond 90 days (*IRC : 18-2000*)(ton)	0	0	0	0	0	0
Net presressing force(after relaxation)(ton)	6370.20	6616.86	6732.41	6827.34	6793.94	6786.58
Stress at C.G. of Cables(t/m^2)	457.38	585.75	678.79	726.08	731.56	731.27
Shrinkage Strain	0.00	0.00	0.00	0.00	0.00	0.00
Creep Strain	0.00	0.00	0.00	0.00	0.00	0.00
Loss due to creep and shrinkage (ton)	428.46	452.63	470.15	479.05	480.09	480.03
Increasing the loss by 20%(ton)	514.15	543.16	564.18	574.86	576.10	576.04
Net prestressing force (after deducting shrinkage and creep) (ton)	5856.04	6073.70	6168.23	6252.48	6217.83	6210.54

Loss due to Creep and Shrinkage in Concrete beyond 90 days						
Stress at C.G. of Cables(t/m^2)	366.90	467.70	542.15	581.31	577.05	584.44
Shrinkage Strain	0.00	0.00	0.00	0.00	0.00	0.00
Creep Strain	0.00	0.00	0.00	0.00	0.00	0.00
Loss due to creep and shrinkage(ton)	507.97	576.98	627.96	654.77	651.86	656.91
Increasing the loss by 20%(ton)	609.56	692.38	753.55	785.73	782.23	788.30
Net prestressing force(after deducting shrinkage and creep)(ton)	5246.48	5381.33	5414.69	5466.75	5435.61	5422.24
Percentage loss(%)	19.11	20.46	21.59	22.04	22.34	22.31

Flexural Stresses

The flexural stresses in the longitudinal direction are evaluated for transfer stage at service stage for different sections.

Stress at top,

σ_t = P/A – P_e/Z_t – M_g/Z_t + M_q/Z_t

Stress at bottom

σ_b = P/A + P_e/Z_b – M_g/Z_b – M_q/Z_b

where,

Mg = Dead load bending moment

Mq = Live load bending moment

Here the summary of the stress are give for both the bottom stress in Table 7.12 and the top stress as in Table 7.13.

Table 7.12: Stress at bottom (t/m^2)

	SEC 1-1	**SEC 2-2**	**SEC 3-3**	**SEC 4-4**	**SEC 5-5**	**SEC 6-6**
Self Weight	0	0	0	0	0	0
Prestress	552.96	723.66	811.21	854.17	859.42	857.1
Immi Loss	540.07	704.53	787.22	827.73	829.97	829.37
Gradual Loss	492.30	646.31	722.01	759.36	760.96	760.34
SIDL	396.70	741.94	817.64	854.99	856.59	855.97
Gradual Loss	340.06	667.72	730.54	761.55	762.91	761.50
LL	53.69	381.27	444.09	475.10	476.45	475.05
50% LL	196.87	524.50	587.32	618.33	619.67	618.27
Diff Temp	-9.78	317.85	380.66	411.67	413.02	411.62
20% Add Loss	30.23	350.96	408.83	437.45	438.02	436.81

Table 7.13: Stress at Top (t/m^2)

	SEC 1-1	**SEC 2-2**	**SEC 3-3**	**SEC 4-4**	**SEC 5-5**	**SEC 6-6**
Self Weight	0	0	0	0	0	0
Prestress	-120.54	46.10	-20.80	-49.49	-55.88	-55.78
Immi Loss	-120.84	48.10	-16.84	-44.56	-50.23	-50.45
Gradual Loss	-121.93	54.18	-6.10	-31.82	-37.00	-37.19
SIDL	-32.52	143.56	83.28	57.54	52.37	52.17
Gradual Loss	-33.81	151.31	97.62	74.95	70.33	70.32
LL	233.99	419.03	365.35	342.68	338.05	338.04
50% LL	100.09	285.17	231.49	208.82	204.19	204.18
Diff Temp	-200.77	-15.69	-69.37	-92.03	-96.66	-96.67
20% Add Loss	233.46	422.20	371.16	349.69	345.42	345.38

Additional Steel Required Due to Differential Temperature Stresses

Here as shown below the additional reinforcement has been calculated for the differential temperature stresses. The data of the property taken are as shown in the Table 7.14.

f_{ck}	45 MPa
f_y	500 MPa
σ_{cb}	15
m = 280/3σ_{cbc}	6.22 M Pa
σ_{st}	240 MPa
K	mσ_{cbc}/(mσ_{cbc}+σ_{st})
0.28	
j = 1-k/3	0.91
A_{st}	= M/σ_{st}jd

Table 7.14: Additional steel for temperature stress

	STRESS t/m^2	Z_t m^4	**BM tm**	**D m**	A_{st} **req** m^2	A_{st} **req** m^2	**Dia cm**	**spac mm**
SEC 1-1	-200.77	9.79	1964.91	2.00	0.05	451.50	3.20	178.13
SEC 2-2	-15.69	9.79	153.53	2.00	0	35.28	1.60	569.95
SEC 3-3	-69.37	9.79	678.95	2.00	0.02	156.01	2.00	201.37
SEC 4-4	-92.04	9.79	900.77	2.00	0.02	206.98	2.00	151.78
SEC 5-5	-96.66	9.79	946.02	2.00	0.02	217.37	2.00	144.52
SEC 6-6	-96.67	9.79	946.15	2.00	0.02	217.41	2.00	144.50

Check For Ultimate Strength

Ultimate Load	=	1.5G+2SG+2.5Q		
(Cl:12 IRC-18)	G = DL BM =		1341.13	kNm
SG = SIDL BM =	875	kNm		
Q = LL BM =	2621	kNm		
Ultimate Load Moment =		**10314.20**	tm	

Ultimate Moment of Resistance

(Cl:13 IRC-18)

Failure by Yield of Steel

M_{ult} = 0.9dbA_s F_p

A_s =	26.60*19		=	505.40	cm^2
db =	200 - 22.27		=	177.73	cm
F_p =	189.7	kg/cm^2			
M_ult =	15335.78	tm	>	10314.20 tm	

Mult = 0.176bdb2f_{ck} + 2/3*0.8*(B_f - b)(db - t/2)tf_{ck}

where,

f_{ck} =	450	kg/cm^2

Failure by Cracking of Concrete without yield of Steel

b =	200	cm		
B_f =	1500	cm		
t =	50	cm		
M_ult =	28829.41	tm	>	10314.20 tm

CHECK FOR SHEAR

Ultimate Shear Force (SF) = 1.25G + 2SG + 2.5Q *(Cl : 12 IRC - 18)*

Net Ultimate Shear Force (V) = Ultimate Shear Force - Loss

Ultimate Shear Resistance V_{co} = 0.67bd$\sqrt{{f_t}^2 + 0.8 f_{cp} f_t}$

(Cl : 14.1.2 IRC - 18)

where, f_y = 50000 t/m^2

f_{ck} = 4500 t/m^2

f_t = 0.24vfck = 16.10 t/m^2

f_{cp} = Compressive stress at centroidal axis due to Prestressing force

Ultimate Shear Resistance V_{cr} = (0.037b$d_b\sqrt{f_{ck}}$ + (Mt/M)*V) *(Cl : 14.1.3 IRC - 18)*

M_t = Cracking Moment = (0.37*f_{ck} + 0.8*f_{pt}) I / y where, f_{pt} = Stress due to Prestress

M = Ultimate Moment = 1.25G + 2SG + 2.5Q

Ultimate Shear Resistance V_{cr} shall not be less than (0.1*b*d*$\sqrt{f_{ck}}$) *(Cl : 14.1.3 IRC - 18)*

A_{sv}/S_v = (V-V_c) / (0.87*$f_y * d_t$)

d_t = Depth from the extreme compression fibre = 1.75 m

P_v = Vertical Component of Prestressing Force.

Here the ultimate shear Reinforcement and ultimate torsion reinforcement are calculated in area/m is found as shown in the Table 7.15.

Table 7.15: Longitidunal shear design

Sec	**1 - 1**	**2 - 2**	**3 - 3**	**4 - 4**	**5- 5**	**6-6**
SF (kN)	1175.5	1175.5	1175.5	1175.5	1175.5	1175.5
P_v **(t)**	1626.43	897.67	564.57	235.64	29.59	0
P_v**(loss) (t)**	1315.6	714.0	442.8	183.7	22.9	0
V (t)	-140.1	461.5	732.8	991.8	1152.5	1175.5
f_{cp} t/m^2	289.7	297	298.8	301.7	300	299.2
V_{co} **(t)**	126.9	128.4	128.8	129.4	129	128.9
db m	1.4	1.64	1.73	1.76	1.77	1.77
f_{pt} t/m^2	584.9	692	750.8	780	781	779.6
M_t **(tm)**	4510.4	5294.5	5725.2	5938.3	5946.91	5935.5
M (tm)	9978.75	9978.75	9978.75	9978.75	9978.75	9978.75
V_{cr} **(t)**	-57.8	250.9	426.8	596.8	693.4	705.8
V_{cr} **(t) (check)**	20.1	20.1	20.1	20.1	20.1	20.1
Final V_{cr} **(t)**	20.12	250.9	426.8	596.8	693.4	705.8
Check	ok	ok	ok	ok	ok	ok
V_c **(t)**	20.121	128.4	128.8	129.4	129.07	128.92
Ult Shear (mm^2/m)	-2105.57	4374.81	7933.8	11328.53	13444.36	13748.16
Tor Shear (mm^2/m)	218.2	218.24	218.24	218.24	218.24	218.24

Check for Ultimate and Torsional Shear Stresses

Table 7.16 shows the check for ultimate and torsion shear stresses for all sections

Table 7.16: Check for stress

Sec	**Shear V(t)**	**Tor** (t)	**b** (m)	d_b	**A** (mm^2)	**t** (m)	**Tor. Shear** (N/mm^2)	**Stress** (N/mm^2)	**Total Stress** (N/mm^2)
1 - 1	-140.16	172.0	1.50	1.48	18.12	0.50	0.128	-0.631	-0.502
2 - 2	461.50	172.0	1.50	1.63	18.12	0.50	0.115	1.877	1.993
3 - 3	732.81	172.0	1.50	1.73	18.12	0.50	0.109	2.823	2.932
4 - 4	991.80	172.0	1.50	1.76	18.12	0.50	0.107	3.741	3.84
5 - 5	1152.52	172.0	1.50	1.77	18.12	0.50	0.106	4.324	4.431
6 - 6	1175.50	172.0	1.50	1.77	18.12	0.50	0.106	4.409	4.516

Cable Elongation Cable elongation is given by:

$\delta = P_{av}.\mathrm{L}/E_s + P_j.L_{grip}/E_s$

where, δ = Cable Elongation

L = Length of Cable = 22800 mm

L_grip = Grip length within Jack = 940 mm

P_j = Jacking Stress = 185.3 kg/mm^2 E_s = Modulus of Elasticity of Steel = 1.95×10^4 kg/mm^2

P_{av} = Average stress after frictional loss

The values for cables elongation at various sections are tabulated in Table 7.17 below:

Table 7.17: Cable elongation

Cable No.	**Avg. Stress** (kg/mm^2)	**Elongation** **(mm)**
1-21	141.48	37.95
22-23	141.43	37.94
24-25	139.75	37.60
26-27	138.03	37.24
28-29	141.43	37.94
30-31	139.75	37.60
32-33	138.03	37.24

Calculation of Minimum Reinforcement

	At Mid	AT End
Width Of Web	500mm	500 mm
Thickness Of Top Slab	500 mm	500 mm
Thickness Of Bottom Slab	500 mm	500 mm
Grade Of Concrete	45 MPa	45 MPa

1) **Minimum Web Steel In Vertical Direction** *IRC 18-2000 CL 15.2*

Min vertical steel in web %	= 0.18	900 mm^2/m Length
	Dia	Spacing
Required steel 10 mm	2 Legged @	174.53 mm c/c
Provided steel 10 mm	2 Legged @	170 mm c/c
A_{st}	provided	923.99 mm^2

2)**M**inimum Web Steel In End Widening *IRC 18-2000 CL 15.2*

Min vertical steel in web %	= 0.18	900 mm^2/m Length
	Dia	Spacing
Required steel 10 mm	2 Legged @	174.53 mm c/c
Provided steel 10 mm	2 Legged @	170 mm c/c
A_{st}	provided	923.99 mm^2

3) **Longitudinal Reinforcement At Centre***IRC 18-2000 CL 15.2*

Min vertical steel in web %	= 0.15	750 mm^2/m Length
	Dia	Spacing
Required steel 10 mm	2 Legged @	209.4453 mm c/c
Provided steel 10 mm	2 Legged @	200 mm c/c
A_{st}	provided	785.4 mm^2

4) **Longitudinal Reinforcement At End** *IRC 18-2000 CL 15.2*

Min vertical steel in web %	= 0.15	750 mm^2/m Length
	Dia	Spacing
Required steel 10 mm	2 Legged @	209.4453 mm c/c
Provided steel 10 mm	2 Legged @	200 mm c/c
A_{st}	provided	785.4 mm^2

5) **Minimum Steel In Top Slab** *IRC 18-2000 CL 15.2*

Min vertical steel in web %	= 0.18	900 mm^2/m Length
	Dia	Spacing
Required steel 10 mm	2 Legged @	174.53 mm c/c
Provided steel 10 mm	2 Legged @	170 mm c/c
A_{st}	provided	923.99 mm^2

6) **Minimum Steel In Bottom Slab** *IRC 18-2000 CL 15.2*

Min vertical steel in web %	= 0.18	900 mm^2/m Length
	Dia	Spacing
Required steel 10 mm	2 Legged @	174.53 mm c/c
Provided steel 10 mm	2 Legged @	170 mm c/c
A_{st}	provided	923.99 mm^2

7) **Minimum Steel In Top Slab At End** *IRC 18-2000 CL 15.2*

Min vertical steel in web %	= 0.18	900 mm^2/m Length
	Dia	Spacing
Required steel 10 mm	2 Legged @	174.53 mm c/c
Provided steel 10 mm	2 Legged @	170 mm c/c
A_{st}	provided	923.99 mm^2

8) **Minimum Steel In Bottom Slab At End** *IRC 18-2000 CL 15.2*

Min vertical steel in web %	= 0.18	900 mm^2/m Length
	Dia	Spacing
Required steel 10 mm	2 Legged @	174.53 mm c/c
Provided steel 10 mm	2 Legged @	170 mm c/c
A_{st}	provided	923.99 mm^2

There reinforcement detailing of the box girder for minimum reinforcement has been shown as in Fig.7.9.

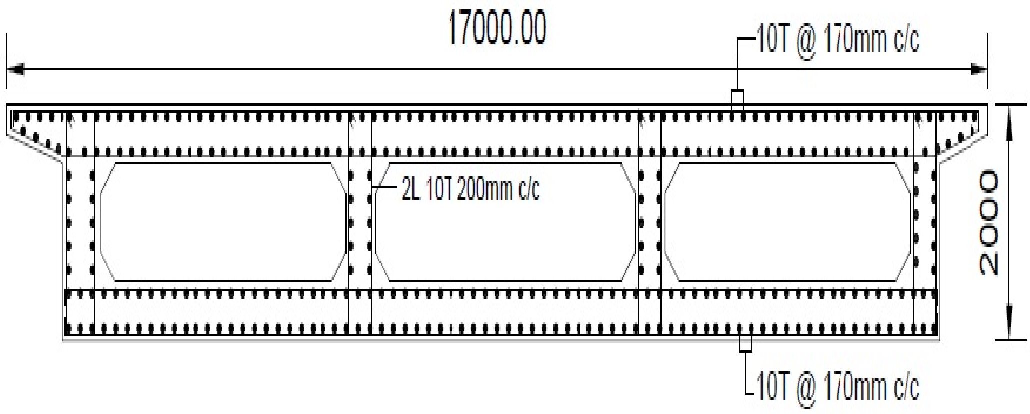


Figure 7.9: Minimum reinforcement detail

7.3 Design of Substructure

The substructure design of the bridge can be done by different type of foundation like well foundation, raft foundation, Piled raft foundation, Pile foundation can be used. The choice of the type of foundation depends on the load it has to resist, geological condition at the site, availability of technology, economical feasibility. The foundation of large structure like cable stayed bridge where large amount of loads are to be resisted has to be properly studied before selecting the type of foundation. The planning of the foundation also becomes an important aspect both for quality and economical point of view. Here the pile group foundation has been designed as discussed below.

7.3.1 Pile Foundation Design

The loads of the bridge can be transfered to medium depths through piles in case of adequate bearing strata is not available at shallow depths. The pile may be bearing pile or friction piles depending on the soil strata.

Here the pile group has been designed for the foundation. The layout of the pile group in plan has been shown as in Fig. 7.10. The pile is designed for biaxial moments, horizontal loads in both direction and vertical loads. The moments and the Horizontal load are equivalent converted into vertical load as discussed below.

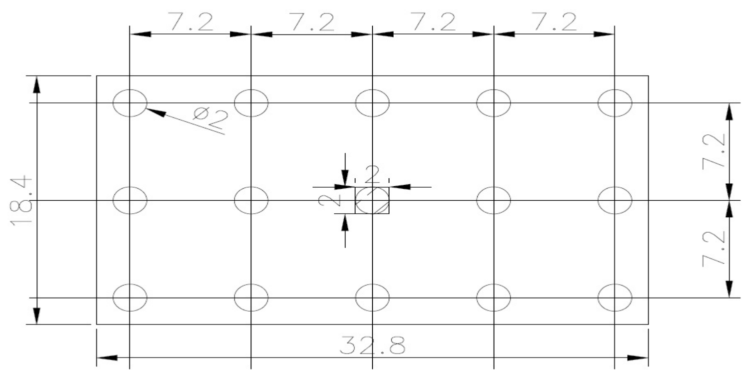


Figure 7.10: Pile layout diagram

Moment Distribution In Pile

DATA

VERTICAL	34502 kN
M_{xx}	19432 kNm
M_{yy}	15299 kNm
F_x	1778 kN
F_y	4990 kN
No of Piles	15no

PILE	rx_i	rz_i	rx_i^2	rz_i^2	M_{xx}	M_{zz}
A1	-14.4	7.2	207.36	51.84	19432	15299
A2	-7.2	7.2	51.84	51.84	19432	15299
A3	0	7.2	0	51.84	19432	15299
A4	7.2	7.2	51.84	51.84	19432	15299
A5	14.4	7.2	207.36	51.84	19432	15299
B1	-14.4	0	207.36	0	19432	15299
B2	-7.2	0	51.84	0	19432	15299
B3	0	0	0	0	19432	15299
B4	7.2	0	51.84	0	19432	15299
B5	14.4	0	207.36	0	19432	15299
C1	-14.4	-7.2	207.36	51.84	19432	15299
C2	-7.2	-7.2	51.84	51.84	19432	15299
C3	0	-7.2	0	51.84	19432	15299
C4	7.2	-7.2	51.84	51.84	19432	15299
C5	14.4	-7.2	207.36	51.84	19432	15299
		Sumation	**1555.2**	**518.4**		

Pile	**P/n**	$\frac{Mxx*rz_i}{r^2z_i}$	$\frac{Mzz*rx_i}{r^2x_i}$	P_i **kN**
A1	2300.13	269.88	-141.65	2428.36
A2	2300.13	269.88	-70.82	2499.19
A3	2300.13	269.88	0	2570.02
A4	2300.13	269.88	70.82	2640.851
A5	2300.13	269.88	141.65	2711.68
B1	2300.13	0	-141.65	2158.47
B2	2300.13	0	-70.82	2229.30
B3	2300.13	0	0	2300.13
B4	2300.13	0	70.82	2370.96

PILE	P/n	$\frac{Mxx*rz_i}{r^2z_i}$	$\frac{Mzz*rx_i}{r^2x_i}$	P_i kN
B5	2300.13	0	141.65	2441.79
C1	2300.13	-269.88	-141.65	1888.58
C2	2300.13	-269.88	-70.827	1959.41
C3	2300.13	-269.88	0	2030.24
C4	2300.13	-269.88	70.82	2101.07
C5	2300.13	-269.88	141.65	2171.90
			Max	2711.68

Distribution Of Loads (Lateral Force)

X Direction

Weight Of Pile Cap

L			32.8 m
B			18.4 m
D			2.65 m
W_t			38383.9 kN
X'			15 m

Angle Between Horizontal And Vertical Load

$tan\theta$			0.046
e			0.122 m
P_i	P/n	P_{ex}'$/\Sigma X'^2$	
	2558.93	20.94	
P_{max}			2579.87 kN
P_{min}			2537.98 kN

Y Direction

Weight Of Pile Cap

L	32.8 m
B	18.4 m
D	2.65 m

W_t			38383.9 kN
Y'			7.2 m
Angle Between Horizontal And Vertical Load			
tanθ			0.13
e			0.344 m
P_i		P/n	P_{ex}'$/\Sigma X'^2$n
		2558.92	122.4
P_{max}			2681.36 kN
P_{min}			2436.48 kN

Max P Load	2300.13 + 27711.7	+2681.36	**7693.18**kN
Bearing Load Capacity			**19232.9**kN

Bearing Capacity Of Pile

Type Of Pile	Bored Pile
Soil Type	Cohesive
Cohesion C @ tip	100 kN/m^2
N_c	9
Dia Of Pile (m)	2 m
Length Of Pile (m)	60 m
Adhesion Co.	0.45
No Of Pile Provided	15
Bearing Capacity Of Pile	
Q_u =	N_cxC_bxA_b + αx$C'_u \times A_s$
	= 2827.43 + 16964.4
	= 19792.03 kN
Safe Bearing Capacity Of Pile	= 7916.81 kN
Total No Of Piles Required	= P/SAFE BEARING CAPACITY
4.4	< 15 **OK**

Total No Of Piles

Considering Biaxial Moment and Axial Load

7693.18 kN <	7916.81 kN	**OK**

Horizontal Bearing Capacity Of Pile

Scour Depth Calculation

Q	6000 m^3/sec	
HFL	110 m	
Bed Level	90 m	
Eff Linear Waterway	=4.8*$Q^{0.5}$	371.81 m
Discharge Per Width D_b	=Q/L	16.14 $m^3/sec/m$
Normal Scour Depth d	=1.34*$(Db^2/Ksf)^{1/3}$	8.56 m
Maxium Scour Depth D	=2d	17.11m
Grip Length	5.70 m	
Base Of Foundation Below HFL	22.82m	
Depth Below Base Level	2.82 m	
IS 2911 PART 2		
E	3.35×10^7	kN/m^2
K	12000	kN/m^2
I	0.78	m^4
R	$(EI/KB)^{0.25}$	5.75
IS 2911	Fig 2	
L_1	2.82	m
L_1/R	0.48	
For Fixed Head Pile		
L_f/R	2	
L_f	11.50	m

Now Lateral Load Capacity

Q	$=12EIy/(L1+Lf)^3$	536.77	kN		
No Of Pile Required		9.29	$<$	15	**OK**
Max Moment In Pile					
M_f	$=Q(L_f+L_1)/2$	3845.28	kNm		
Reduction Factor		m	$= 0.5$		
Max Moment		$= mM_f$	1922.64 kNm		
No Of Pile Required		10.10	$<$	15	**OK**

Settlement Check

Density			19 kN/m^3
Depth of soil			60 m
length of pile raft			32.8 m
Width of pile raft			18.4 m
C_c			0.1
e_0			0.9
S	$C_c H/(1+e_0) \times \log(P_0+\Delta P)/P_0$		
L_e			$=2/3\times L$
h_1			40 m
h_2			20 m
h_3			20 m
P_0	$= \gamma\times d$		950 kN/m^2
	Dispersion	1H:2V	
	b	25.4 m	
$\Delta P=$	$P/(b+d)^2$		6.07 kN/m^2
S	$C_c H/(1+e_0) \times \log(P_0+\Delta P)/P_0$		
			7.28 mm

P_0	$= \gamma \times$d	1330.33	kN/m^2	
	b	45.4m		
ΔP=	P/$(b+d)^2$		2.59	kN/m^2
S	C_cH/(1+e_0) $\times$ log(P_0+ΔP)/P_0			
		3.11 mm		
Total settlement	10.39mm	>	50mm	
			OK	

Design of pile

Total Vertical Load		7693.18 kN	
Dia Of Pile (m)		2000 mm	
Length Of Pile (m)		60000 mm	
f_{ck}		45 MPa	
f_y		500 MPa	
No of pile		15	
cover		100 mm	
L_{eff}		40000 mm	
L_{eff}/D	20	Long Column	
	>		
	12		
e_{min}	=L/500+D/30	78.67 >	20
M_u		60.52 kNm	

Since bored piles are provided there will be no handling stresses

Correction factor for long columns	=1.25-L_e/48B
	0.83
Design moment	72.62 kNm
Design Load	923.18 kN
d'/D	0.05
P_u/f_{ck}bD	0.043
$M_u/f_{ck}bD^2$	0.021

SP 16	**CHART**	**59**		

As per interaction Dia. Amount of reinforcement is less

Nominal rcc

A_{st}	min	25132.74	mm^2	
	8%			

Provide 32 mm Dia of 32 nos

Lateral Ties

Minimum volume of Lateral reinforcement/L (m) of pile

A_{st}	min	6283185.31	mm^2	
	2%			
Vol of tie of 8 mm Dia		284244.61	mm^3	
No of Tie/meter of pile		22.11		
Spacing of Tie/meter of pile			452.39	mm c/c

Hence spacing for Lateral reinforcement

			452.39 mm
		$<$	600 mm
		$>$	384 mm
		$<$	512 mm

Provide spacing for lateral reinforcement 380 mm c/c

The foundation has been designed as discussed above. The design is done by considering the moments in both direction and lateral loads in both direction with vertical loads in addition. All the moments and the horizontal loads are distributed and the maximum vertical loads are consider for the design. The settlement check is also done. The detailing of the pile cross-section is as shown in the Fig. 7.11.

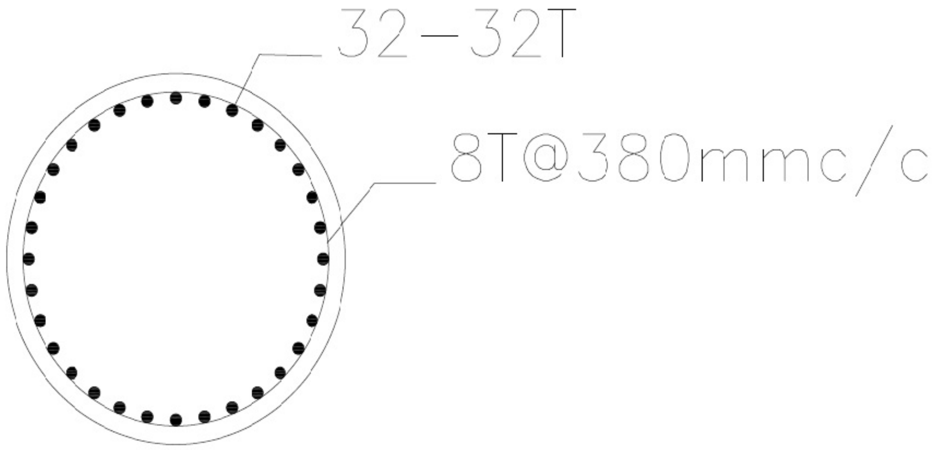


Figure 7.11: Pile reinforcement detail

Design of Pile Cap

Pile Cap Dimension

B	32.8	m	
D	18.4	m	
Load	34502		
No of piles	15		
Column dim			
B	2000	mm	
D	2000	mm	
f_{ck}	45	MPa	
f_y	500	MPa	
Bending moment at the face of column			
=	6261.47	kNm/m width of pile	
Factored moment	9392.21	kNm	
Eff depth d	1252.71	mm	
D	1300	mm	
Punching shear force		51753	kN
Punching shear stress	3.02	> 1.1	MPa
		NOT OK	

Depth required from punching shear consideration

	11762045	=	$d_2 + 2000 \times d$		
	solving we get				
d =		2573	mm		
D =		2650	mm		

Reinforcement

M_u = 0.87× f_y × A_{st}(d-$f_y A_{st}/f_{ck}$ × b)

9392.21 = (1119255 × A_{st}-4.83$A_{st}{}^2$)

		Solving			
	A_{st}		8720	mm^2	
Provide	32 mm bar	spacing	92.2 mm	c/c both	direction
Provide	32 mm bar	spacing	90 mm	c/c both	direction
	A_{st} prv			8933	

Check for one way shear

S.F/m width of section			958.39 kN		
Shear stress			0.37 MPa		
100A_{st}/bd			0.35		
τ_c	IS	456	0.43 MPa	>	0.37 MPa

OK

The Pile cap also designed as discussed above. The settlement check of the pile foundation is done and is coming with in safe limit. The bored cast in-situ pile is designed as slender column form the design aids SP 16 .The reinforcement detailing is done in Auto-CAD package. The reinforcement detail of the pile reinforcement layout is shown in Fig. 7.12. The section detail of the pile cap with pile and column junctions are shown in Fig. 7.13 and Fig. 7.14.

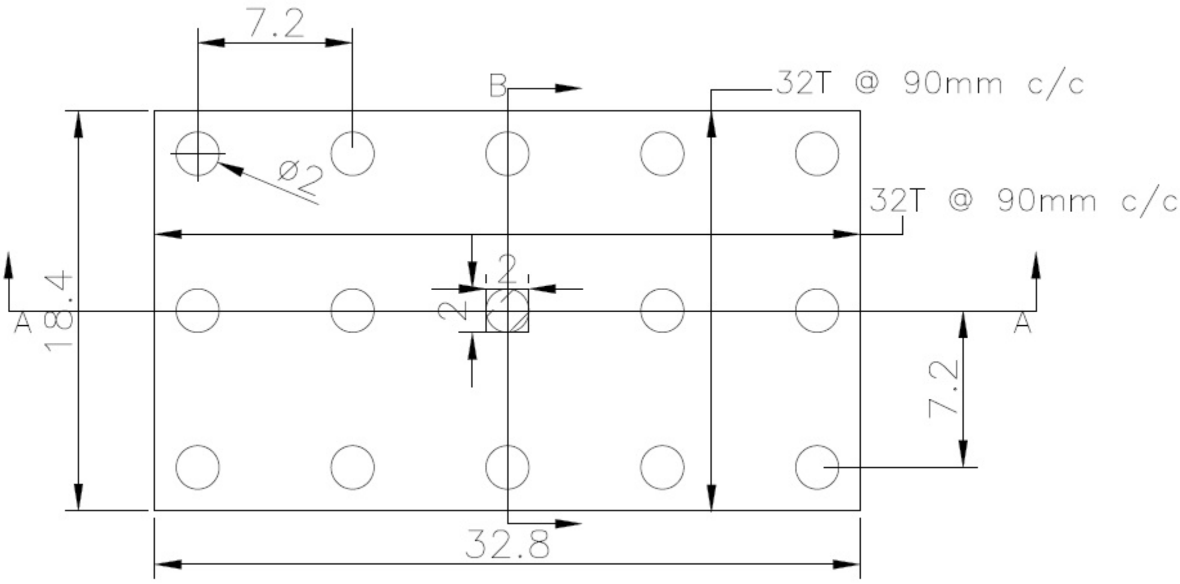


Figure 7.12: Pile raft detail plan

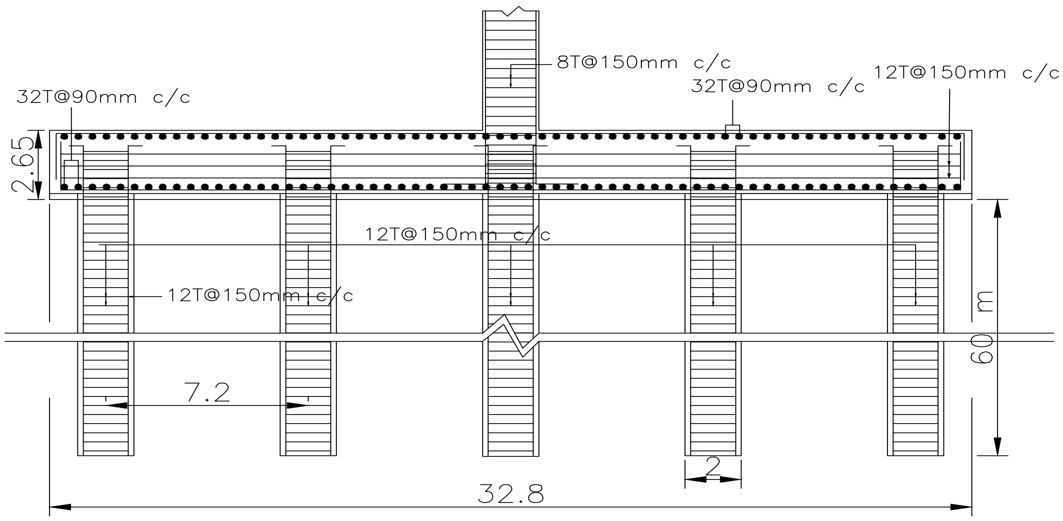


Figure 7.13: Pile raft detail section AA

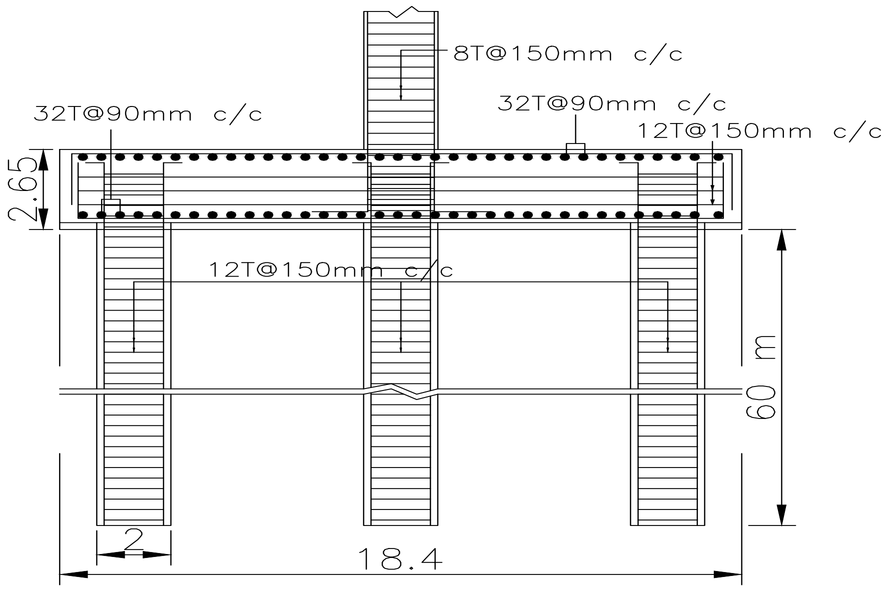


Figure 7.14: Pile raft detail Section BB

Chapter 8

Summary and Conclusion

8.1 Summary

Here study of the cable stayed bridge is done for its various Historical facts, various components of the bridges, behaviour of the cable stayed bridge, types of the pylon arrangements, type of the cable configuration arrangements, Type of material used for Cable, deck. The classification is also done according to shape of the pylons and the Geometrical configuration of the cables has been done to give a brief idea about cable stayed bridge.

Literature review has been done to understand the current trends of research of work going on cable stayed bridge. The literature survey was limited to the basic understand of the cable stayed bridge under Dead Load, Live Load and on some advanced topics like the Aerostatic Effect and the Dynamic Response of the bridge.

For validation of the software used an example solved in a technical paper has taken and verified by using SAP 2000 software.Cable stayed bridge of 200m span having 100m central span with H type pylon shape and with semi-harp type cable configuration has been taken for analysis in SAP2000.

The cable stayed bridge is modeled with proper technique from the guidance of various literature surveyed. Then study for the behavior under various loading is undertaken.

Live load was taken according to IRC 6:2000. IRC Class A and Class 70R vehicle load along with Aerostatic wind loads was undertaken.A Dynamic analysis in the form of Linear Time-history is also carried out to investigate dynamic response of the Cable-stayed Bridges and various response quantities such as Bending-moment, Shear-force,Torsion and Axial force are represented.

Parametric Study has been carried out by varying the shape of the pylon and the central span of the bridge.The shape here taken for study are H shape, A shape, Diamond shape, Delta shape, Inverted Y shape. The span taken for study varied from 200m,400m,600m,800m. The Linear and Dynamic case has been compared for the response quantities.The result of study is represented in graphical form for various response quantities like Bending-moment, Shear-force, Torsion and Axial force.Findings of the study were the trend line remains more or less same in both cases for axial force for cables, girder and pylons. The performance of different shape are better for different response quantities so it cannot be merely concluded form analysis which is better so a detailed cost comparison can only give the better shape suitable by designing each shape and components.

Design of each components like Cable, Pylon, Box Girder and Foundation has been carried out for 200 m H type Cable stayed bridge. The cable are designed by taking maximum axial force acting on it and the providing the number of cables required for resisting the force. Pylon are designed as slender column for maximum biaxial moment and axial force . Box Girder is designed as Pre-Tensioned three cell Box Girder for the maximum forces obtained from analysis. Pile foundation is provided as foundation to resist both the vertical and horizontal forces.

8.2 Conclusion

- The increase in span increases the axial tensile force in the cable.This id because the segment length and depth of the deck increases so the weight of the deck increase and thus the axial force in the cables.

- The Delta shape of pylon has lesser force than others in both Linear and Dynamic case for Girder axial force while diamond performs better for shear. For girder Torsion A shape pylon is better than others.
- For pylon Axial forces the Diamond shape Performance has been better with increase in span, in shear and Pylon Torsion both H type and A type are shaped pylons are better while in Pylon moment the Inverted Y type along with H type and A type shows better performance.
- The performance of different shape are better for different response quantities so it cannot be concluded form this analysis which is better so a detailed cost comparison can only give the better shape suitable.
- Prestressed Box Girder with Multi cell are designed considering the maximum moments for all the combination for vertical and lateral loads.
- Due to Heavy Vertical and Lateral Loading Pile foundation are required of diameter of 2m upto the depth of 60m. Total 15 no of Pile are Provided.

8.3 Future Scope

- Study for various geometrical arrangements of cables like fan type, harp type, mixed type, star type of cables configurations to the varying Central Span.
- Comparison between various shapes like H-type, inverted y type, portal type, diamond shape type of pylons used to the Cable geometrical configuration arrangement.
- Study of effect of number of cables and length of central panel.
- Study of the Pylon height to central span length ratio.
- Design of all shapes of the Pylons and their cost comparison to find the best performing shape of pylon.

- Comparison of Cable Stayed Bridge with Suspension Bridge.
- Analysis of Hybrid type of Cable Stayed Bridge.
- Non Linear effect of cables of the cable stayed bridge is important for study.
- Soil interaction of Cable Stayed Bridge can be studied.
- Oscillation of cable due to Wind and Rain can also be Studied.
- Wind tunnel experiment can be carried on the cable stayed bridge for fluttering and buffeting analysis of cable stayed bridge.
- By varying the different type of Deck Girder cost comparison for best suitable deck can be proposed after studying

Appendix A

List of Useful Websites

- http://civilengineer.webinfolist.com/research/dissert.html
- http://ntlsearch.bts.gov/tris/record/tris/00490456.html
- http://en.wikipedia.org/wiki/List_of_largest_cable-stayed_bridges
- http://dspace.mit.edu/handle/1721.1/32550?show=full
- http://www.ebooknetworking.net/ebooks/cable-stayed-bridges.html
- http://www.springerlink.com/content/6u46melrufh8w73u/
- http://www.roadtraffic-technology.com/projects/bandra/
- http://www.nbmcw.com/articles/bridges/

Appendix B

List of Paper Published/Communicated

List of Paper Published

- Blesson Thomas B. and Sonal Thakkar P. "Aerostatic and Dynamic Effect on Different Shapes of Pylons of Cable Stayed Bridge, National Civil Engineering Student's Symposium AAKAAR-2010, Department of Civil Engineering, IIT Bombay, 27-28 February 2010. **(Won second prize in PG category)**

List of Paper Communicated

- Blesson Thomas B. and Sonal Thakkar P., "Dynamic Effect on Different Shapes of Pylons and span of Cable Stayed Bridge", Innovative World of Structural Engineering (ICIWSE-2010), Aurangabad, 17-19 September 2010. (Abstract accepted)

- Blesson Thomas B. and Sonal Thakkar P., "Aerostatic and Dynamic Effect on Different Shapes of Pylons and Span of Cable Stayed Bridge", The 5th Civil Engineering Conference in the Asian Region and Australasian Structural Engineering Conference (CECAR-2010), Sydney, Australia, 8-12 August 2010. (Abstract accepted)

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[10] IS: 456:2000,*Plain and reinforced concrete code of practice*,Bureau of Indian Standards, New Delhi.

[11] IS: 6:2000*Standard Specification and code of practice for Road Bridges sec-II Loads and Stresses*, The Indian Roads Congress,New Delhi

[12] IS: 21:2000*Standard Specification and code of practice for Road Bridges sec-III, Cement concrete(Plain and Reinforced)*,The Indian Roads Congress,New Delhi

[13] IS: 18:2000*Design Criteria for Prestressed Concrete Road Bridges (Post Tensioned Concrete)*,The Indian Roads Congress,New Delhi

[14] SP: 16*Design Aids For Reinforced Concrete to IS:456-1978*,Bureau of Indian Standards, New Delhi.

[15] IS:2911 part 2:1979 *Code of "practice for Design and construction of Pile foundation sec II Bored Cast-in-Situ Piles*,Bureau of Indian Standards, New Delhi.

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