Analysis and Design of Cable-Stayed Bridge and Cost Comparison with Typical Bridge

Dissertation

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CERTIFICATE

This is to certify that the Major Project entitled **"Analysis, Design of Cable Stayed Bridge and Cost Comparison with Typical Bridge"** submitted by **Mr. Rahil M. Lohia (03MCL05)**, towards the partial fulfillment of the requirements for the award of degree of **Master of Technology (CIVIL)** in field of **Computer Aided Structural Analysis and Design (CASAD)** of Nirma University of Science and Technology is the record of work carried out by him under my supervision and guidance. The work submitted has in my opinion reached a level required for being accepted for examination. The results embodied in this dissertation, to the best of my knowledge have not been submitted to any other university or institution for award of any degree or diploma.

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ABSTRACT

In the recent years cable stayed bridges have received more attention than any other bridge mainly due to their ability to cover large spans. Cable stayed bridges have proved to be emerging option to negotiate large valleys of water ways. Due to superiority and less number of substructures, propagation and development of cable stayed bridges has increases substantially. Options of cable stayed bridge for road over bridge in city limit is also assessed and a few of th projects are implemented and completed. With the development of material technology, spans which were considered impractical are now easily negotiated. Cable stayed bridge technology has grown along with the growth of material technology and construction technology. Examples like Milau viaduct, Tatatra bridge of Japan are few examples of limit that can be easily reached by cable stayed bridges. In India few of the cable stayed bridges are constructed and a couple of them are underway. Second Hoogly Bridge is 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 at Nagpur.

Cable Stayed bridge technology is relatively new in India and designers are still reluctant about cable stayed alternate. The main cause is the cost factor which in general perception is very high. The cost of cables is actually very high as compared to other materials used in construction. Moreover it requires anchorages complication that results in increased cost. However high cost of cables is nullified by less number of substructures and reduced thickness of deck, lowering the cost of bridge. The contractors for such specialized jobs are very limited in India. But increase in number of cable stayed bridges can give wide exposure in this field which will ultimately lower the construction cost.

An attempt has been made to investigate the cost of a cable stayed bridge. A span of 288 mts (overall), 160 mts central span and a side span of 64 mts on either sides for the construction of cable stayed bridge. Analysis of the bridge is carried on SAP-2000 followed by design and costing of the bridge components. The cost of cable stayed bridge is compared with conventional type of bridge assumed to be constructed on the same span. A conclusion on the cost of cable stayed bridge is drawn at the end of study.

The first Chapter consists of introduction to the cable stayed bridge, various components of the bridge, methods of construction, protection, types of stays etc.

Second Chapter introduces to the methods of analysis linear and nonlinear analysis, dynamic analysis, iterative methods, seismic analysis and use of computers for the analysis.

Third Chapter is the literature review for the cable stayed bridge with various literatures on analysis and design of the bridge, various parameters of the bridge design and various effects on the bridge.

Fourth Chapter discusses the use of SAP- 2000 software for the analysis of bridge structure. A beginning to end of the procedure of analysis of the structure using SAP-2000. Discussed in detail are various aspects of modeling and analysis for the analysis to be carried out.

Fifth Chapter is the analysis result and design of various components like cable stays, pylon, deck, pile group and pile cap.

Sixth Chapter consists of quantity sheets of various components designed in previous chapter and the abstract sheet of the quantities of various components.

Seventh Chapter discusses the parametric study carried for various bridges with various configurations. Bridges with varying height of pylon 24 m, 32 m, 40 m and cables 24 nos., 30 nos. and 36 cables are taken and results for the for axial force and bending moment in pylon cable and deck are discussed. Further more is discussed the comparison of the cost of cables, pylon and deck.

Eighth Chapter is the comparison of cost of cable stayed bridge for study with the conventional I-girder Prestressed bridge.

Nineth Chapter discusses the conclusion and future scope of work for the topic.

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CHAPTER 1
INTRODUCTION

1.1 GENERAL

During the past decade Cable-Stayed Bridges have found wide applications because of the benefit of obtaining optimum structural performance from material like concrete and the steel. Cable stayed bridges are constructed along a structural system which comprises an deck and continuous girders which are supported by stays, i.e. Inclined cables passing over or attached to towers located at the main piers.

The idea of using cables to support bridge spans is by no means new and a number of examples of this type of construction were recorded a long time ago. Unfortunately, the system in general met with little success, due to the fact that the behavior was not fully understood and that unsuitable materials such as bars and chains were use to form the inclined supports or stays.

Wide and successful application of cable-stayed systems was realized only recently, with the introduction of high-strength steel, development of welding technology and progress in structural analysis. 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 statical analysis of their three-dimensional performance.

Cable has played a decisive role in the development of cable stayed bridge technology. The recent ascendancy is primarily due to the development of reliable high strength steels for the cables.

The structural principle behind a cable-stayed structure is simple. It is essentially a multiple triangulated force system. Applied loads produce tension in the cable, which is balanced by compression within the deck and tower. However there are secondary effects like change in position of the applied loads, the distribution of loads and general deformation of structure. These secondary effects can modify the axial forces and introduce significant bending moments and shears into the system.

Essentially there are two forms of cable-stayed bridges:

1) Symmetrical having a pattern of cables symmetrical about the mid span and hence having towers at each end of the main span.

2) Asymmetrical having a single tower with a back span generally shorter than the main span.

Stiffness of the overall structure is provided by;

1) Stiff towers either stiff in it through having an inverted V form along the axis of the bridge or stiffened by taking backstays to individual.
2) A stiff anchor span generally achieved by employing intermediate tension piers.
3) A combination of the stiffness of the main span, the tower and the back span providing the greater contribution by virtue of its reduced span/depth ratio.

Wide and successful application of cable-stayed system has gained momentum because of the following reasons:

1) Introduction of high strength steel
2) Development of welding techniques
3) Progress in structural analysis
4) Development and application of electronic computers for precise statical analysis for three-dimensional performance

1.2 EVOLUTION OF CONCEPTUAL DESIGN

The cable stayed bridge follow basically to lines of development distinct from primitive bridges: first; cord bridges, and second; wooden bridges.

Modern cable stayed bridges took all that suspension bridges could possibly offer. However, their structure being self anchored they present a few peculiar problems. The first of them is that only the vertical components of force in the cables are used to support the bridge. This evidently requires cables to be as near as possible, so that a better use is obtained of their force. Another problem lies in the fact that a large number of cables are needed for great spans; it is therefore necessary to construct higher and higher towers in order to obtain largest components in the cable vertical.

It is logical that there is a physical limit to the height of the tower, resulting among other problems, from rigidity it must present to buckling.

Also it should be noted that the possibility of constructing very wide decks generates a possibility of great strain on the transversal direction of it. This problem is met by introducing a very large number of cross girders.

1.3 REVIEW OF BRIDGE

The recent surge in the use of cable-stayed bridges is due mainly to the fact that they offer the opportunity to cross-large obstacles with elegance and economy. However this type of design is equally suitable for small and medium-span structures, which are by far the most numerous. The reasons, which can lead to the choice of cable-stayed solution, are widespread. One of the more widely appreciated is linked to the clearance below the deck, which is often restricted. The use of this type of structure is then naturally advantageous over classic through type bridges, lattice girder and arches. The technical progress in general and that of modern constructional methods in particular have ensured that cable stayed bridges have become more and more economical and competitive for spans of over 50m in length.

1.4 STRUCTURAL SYSTEM

The structural system for cable-stayed bridges consists of four components viz.

1) The cable system, which supports the deck system.
2) Deck system comprising of stiffening girder and deck slab, which supports the traffic.
3) The main towers, which support the cable system and therefore carry most of the load, vertical or horizontal.
4) The end foundation, which carries the vertical and horizontal forces from the cable in case of suspension bridges. In the case of stayed bridge, the end foundations primarily carry the unbalance loads.

1.4.1 Structural Advantages

The introduction of the cable-stayed system in bridge engineering has resulted in the creation of new type of structures, which possess many excellent characteristic advantages. Outstanding among these are their structural characteristics, efficiency and wide range of application. The basic structural characteristics and reasons for the rapid development and success of cable-stayed bridge are outlined below:

1) Cable stayed bridges present a space system, consisting of stiffening girders: steel or concrete and supporting parts as towers acting in compression and inclined cables in

tension. By their structural behavior cable stayed systems occupy a middle position between the girder type and suspension type bridge.

2) The main structural characteristic of this system is the integral action of the stiffening girder and prestressed or post tensioned inclined cables, which run from the tower tops down to the anchor points at the stiffening girders. The girders take horizontal compressive forces due to the cable action and no massive anchorages are required. The substructure therefore is very economical.
3) The anchor forces at the ends of the cable-Stayed bridge act vertically and can usually be balanced by the weight of the pier and its foundation without much additional cost.
4) Introduction of the orthotropic system has resulted in the creation of new types of superstructure, which can easily carry the horizontal thrust of stay cables with almost no additional material.
5) With orthotropic type deck the stiffened plate with its large cross sectional area acts not only as the upper chord of the main girder and of the transverse beam, but also as the horizontal plate girder against wind forces, giving modern bridges much more lateral stiffness than the wind bracings used in old systems.
6) Another structural characteristic of this system is that it is geometrically unchangeable under any load position on the bridge, and all cables are always in a state of tension. This characteristic of the cable-stayed systems permits them to be built from relatively light flexible elements.
7) The important characteristics of such a three dimensional bridge is the full participation of the transverse structural parts in the work of the main structure in the longitudinal direction. This means a considerable increase in the moment of inertia of the construction, which permits a reduction of the depth of the girders and a consequent saving in steel.
8) The introduction of the cable-stayed system has greatly improved the aesthetic quality of bridges. The design for such structures offers smooth flowing with the natural surroundings.

1.4.2 Advantage of Concrete Cable-Stayed Bridges

1) The main girder can be very shallow with respect to the span. Span-to-girder depth ratio varies from 45 to 100. With proper aerodynamic streamlining and multistays the

deck structure can be slim, having span-to-depth ratio of 150 to 400, and not convey a massive visual impression.

2) Concrete deck structures, by virtue of their mass and because concrete has inherently favorable damping characteristics, is not as susceptible to aerodynamic vibrations.
3) The horizontal component of cable stay force, which causes compression with bending in the deck structure, favors a concrete deck structure. The stay forces produce a prestressed force in the concrete and concrete is at its best in compression.
4) The amount of steel required in the stays is comparatively small. A proper choice of height of pylon with respect to span can yield an optimum solution.
5) Erection of the superstructure and cable stays is relatively easy with today's technology of prestressing, prefabrication and segmental cantilever construction.

1.5 FACTORS THAT GOVERN PROPAGATION OF CABLE-STAYED BRIDGES

Cable-Stayed bridge fulfills the basic requirements of selection namely:

1) Aesthetics
2) Environment
3) Navigational requirements
4) Durability and Economy

1.5.1 Aesthetics

The Cable stayed bridge integrates with the city lines in view of the facts that pylons and stays would almost resemble and close reproduction of mast and sail of the fishing vessels and this adds excitement to the natural skyline. The architectural elements covering pylon, footpath, superstructure, deck profile, its sleek appearance, lighting, concealment of utility services etc. provides very good aesthetics.

1.5.2 Environment

Environment that may get affected by construction of long span bridge of importance is important to consider. In cable stayed bridges in view of its pier-free, long span, lesser interferences would be there and hence preferred.

1.5.3 Navigational Requirements

Cable stayed bridge gives large navigational room and results into a better solution for navigational rivers.

1.5.4 Sub-Soil Investigation

Foundation design gets based on the sub-soil investigation and depending on difficulties in achieving necessary bearing, weight ages have to be given for different types of foundations and number of foundations for each alternative. Cable stayed due to less no. of piers has an advantage over other types of bridges.

1.5.5 Durability

This is an important factor, more so in case where public funds are involved. Due consideration to marine/highly corrosive environment need be given in selection of suitable materials and in specifying the anti corrosive treatment and other care for longer durability.

1.6 CABLE STAYED BRIDGES IN INDIA

Cable stayed bridges have come to the Indian scene recently, a pedestrian bridge at Roorkee- having been completed in 1981. Several other bridges have come up, a major bridge being the six lane second Hoogly Bridge at Calcutta having 457m spans.

Engineers in India were first exposed to the idea of using cable-stayed bridges in the 1960s. An earnest attempt was made to construct a multi-span cable stayed bridge of concrete across the Ganga River at Allahabad. Several bridges have since been considered and have progressed to different stages, starting from feasibility study to completeness.

Following are the few examples of major cable stayed bridges in India:

Sr.	Bridge	Span
1.	Bridge over Ganga canal at Roorkee	71.2 m Central Span
2.	Concrete Bridge at haridwar	130m Central Span
3.	Concrete bridge at Akkar in Sikkim	154 m Span (Single Pylon)
4.	Over Bridge at Krishnarajapuram, Banglore	230m Central Span
5.	Anegundi Bridge	260m Central Span
6.	Yamuna Bridge at Allahabad/Naini, Uttar Pradesh	390m Central Span

7.	Second Hoogly Bridge at Calcutta	457.2m Central Span
8.	Bandra Worli Sea Link Project	600m Central Span

Table 1.1 Cable Stayed Bridges in India

1.7 COMPONENTS OF A CABLE-STAYED BRIDGE

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.

The various components of a cable-stayed bridge are:

1) Pylons or Towers
2) Cable system supporting the deck
3) Deck System

1.7.1 Pylon or Tower

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 fig., which shows that they may take the form of:

1. H-Type or Twin Tower
2. Portal Tower
3. Inverted Y- Type
4. Single Tower
5. Diamond Tower

The trapezoidal portal frames and twin towers are suitable for two vertical plane system, however, trapezoidal portal frames gives more transverse stability to the bridge compared to twin towers. A-frame tower are suitable for two inclined plane systems, whereas the single towers are suitable for one plane system. In case of the side towers the deck is on one side of the pylon therefore they are subjected to a large amount of bending moments by cantilever action.

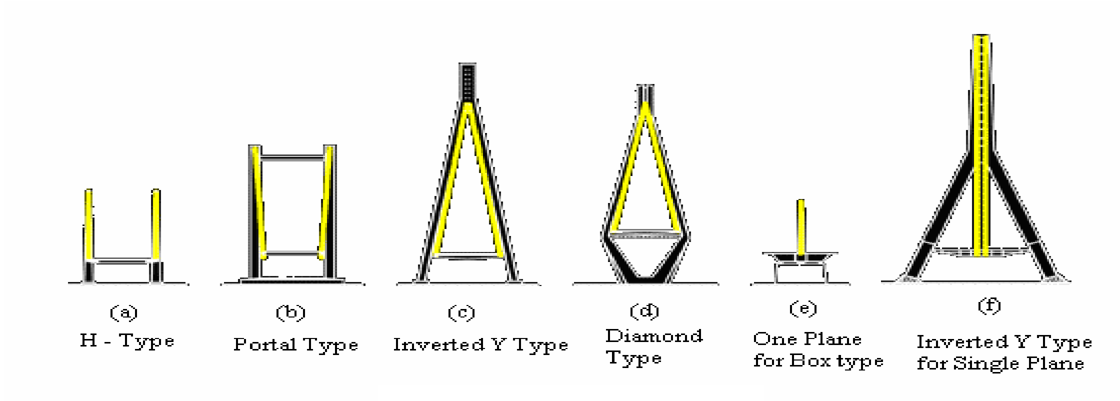


Fig 1.1 Types of Pylon

There are three different solutions possible regarding the support arrangement of the tower:

1.7.1.1 Tower fixed at the foundation

In this case large bending moments are produced in the tower. The majority of the cable-stayed bridges in Germany are built with tower fixed at the base. The disadvantage of high bending moment is offset by increased rigidity of the structure.

1.7.1.2 Tower fixed at the superstructure

In the case of single-box main bridge system, the towers are generally fixed to the box. With this arrangement it is necessary not only to reinforce box but to provide strong bearings. The supports also may resist the additional horizontal forces caused by the increased friction forces in the bearings.

1.7.1.3 Hinged Tower

For structural reasons, the towers may be hinged at the base in the longitudinal direction of the bridge. This arrangement reduces the bending moments in the towers and the number of redundant, which simplifies analysis of the overall structure. Also, in case with bad soil conditions, linear hinges at the tower supports are provided, allowing longitudinal rotation, so that bending moments are not carried by the foundation.

The height of the tower is determined from several considerations such as cable arrangement, visual appearance, economics etc. The recommended height of tower is between 20-25% of the central span length.

1.7.2 Cable System Supporting the Deck

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

1.7.2.1 Types of Cable Arrangement

According to various longitudinal cable arrangements cable stayed bridges could be divided into the following basic systems of arrangement:

1. Pure Fan Type
2. Harp Type
3. Modified Fan Type

1.7.2.2 Pure Fan Type System

In this system all cables are concentration to the top of the tower. Structurally, this is the most efficient system, as by taking all cable to the tower top the maximum inclination to the horizontal is achieved and hence it increases the load-balancing component of the cable tension, at the same time decreases the axial force in the deck. The horizontal force introduced by the cable in the deck is less. Longitudinal bending of the pylons remains moderate. It is not only possible but also necessary to select side spans which are less than half the central span in length. Where erection of the structure is by corbelling out, it is possible to take advantage of the stability provided by the piers or the abutments well before the closing of the central span.

Movements of the deck due to changes in temperature can be absorbed by conventional expansion joints placed across the abutments, the horizontal connection between the pylons and the deck is freed. The deck-to-pylon connection provided by the stays is, in fact too flexible to develop critical force systems.

The flexibility of the structure is favorable where horizontal movements of the deck take place and increases the stability against seismic activity.

The high-capacity back-stay cables, anchored through the first piers or the abutments, reduce the deflections of the pylons and the deck.

An ideal convergence cannot take place in practice, and for this reason it is necessary to spread the anchorages to a greater or lesser extent, depending on the geometry and size of the work. In spite of this spread of forces, the heavily stressed zone can generally be constructed only by methods which are complicated, costly and often far from elegant.

The primary force in the tower is the axial compression; the bending of the tower is minimized. However, where a number of cables are taken to the top of the tower, the cable supports within the tower may be very congested and highly stressed zone, so the construction and the detailing becomes very complex in that region.

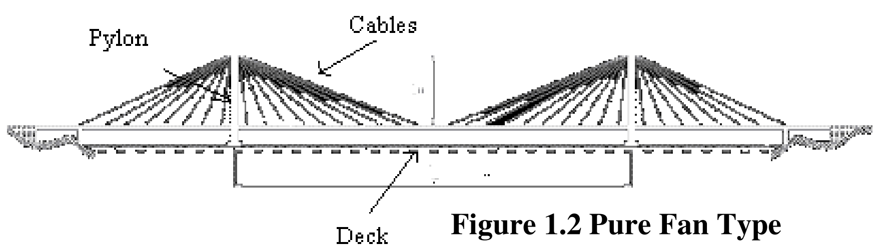


Figure 1.2 Pure Fan Type

1.7.2.3 Harp Type Arrangement

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

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

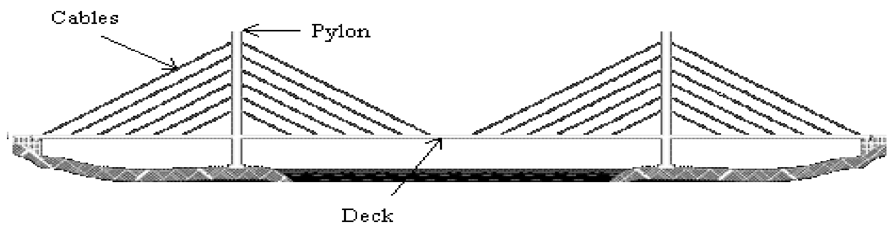


Figure 1.3 Harp Type

1.7.2.4 Modified Fan Type Arrangement

The intermediate solution, between the extreme harp and fan pattern, makes it possible to combine in a satisfactory manner the advantages of both these systems, whilst avoiding their disadvantages. A Modified fan type configuration has shown itself to be ideal, and large numbers of modern cable stayed bridges have been built using this principle.

By spreading of the stays in the upper part of the pylon, good design of the anchorage details is possible, without appreciable reduction of the depth and hence the efficacy of appreciable reduction of the depth and hence the efficiency of the stay system. The situated close to the pylon are more steeply inclined than those in harp pattern, which makes it possible to reduce the stiffness of the horizontal connection between the pylons and the deck- a stiffness which can itself be disadvantageous. With the aim of simplifying the anchoring of the first stay in the pylon and for aesthetic reasons the first bay is generally somewhat larger than the normal cable spacing through out the bridge.

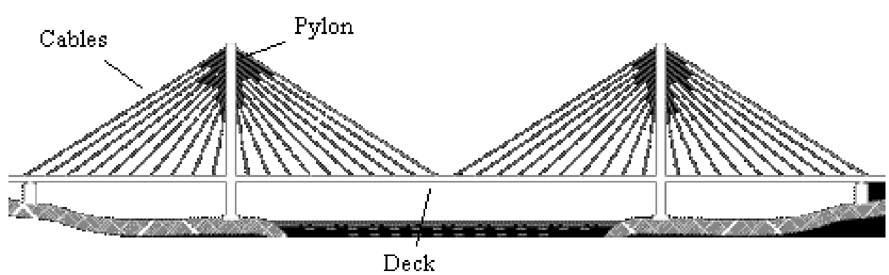


Figure 1.4 Modified Fan Type

1.7.3 TRANSVERSE ARRANGEMENT OF CABLES

With respect to the various positions in space, which may be adopted for the planes in which the cable stays are disposed, there are two basic arrangements as show:

1.7.3.1 Single Plane System

In this type of system, there is only one vertical plane of stay cables along the middle longitudinal axis of the superstructure. In this case the cables are located in a single vertical strip, which is not being used by the traffic. The single plane system also creates a lane separation as a natural continuation of the highway approaches to bridge. This is an economical and aesthetically acceptable solution, providing an unobstructed view from the bridge similarly the bystander looking diagonally at the bridge does not see a double line of the cables. In addition this system also offers the advantage of relatively slender pylons, because their size is determined by the width of the main girder. All the possible variations regarding the longitudinal arrangement of cable are applied to single plane system.

This arrangement requires relatively heavy box-girder with considerable torsional rigidity in order to resist the eccentric live load. The torsional rigidity on other hand increases the aerodynamic stability of the deck.

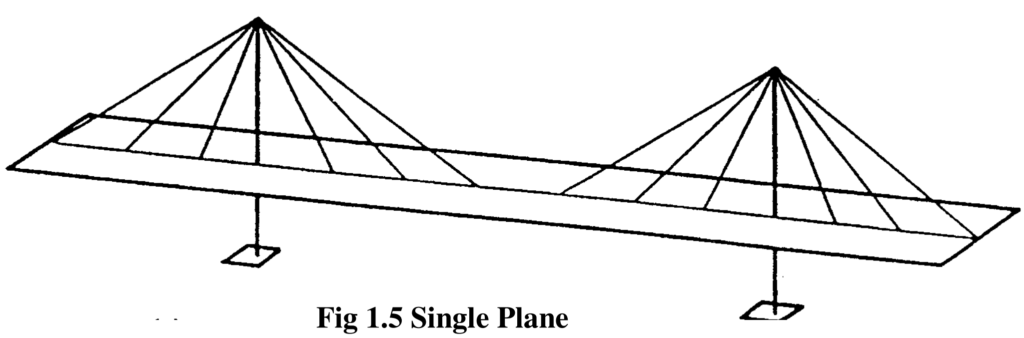


Fig 1.5 Single Plane

1.7.3.2 Two Vertical Plane Systems

This type of system consists of two planes of stay cables placed vertically on each side of the deck. Two plane systems is aerodynamically more stable compared to one plane system, the type of pylon suitable for this type of system is H-Shaped vertical tower.

In two-plane system, the cable-anchorage at the deck-end can be placed either inside or outdid the bridge rallying. In former case, a dead space is created which cannot be used to carry the traffic. In later case, auxiliary construction is needed to transfer the cable force from outside the railing to the load bearing portions of the cross-section.

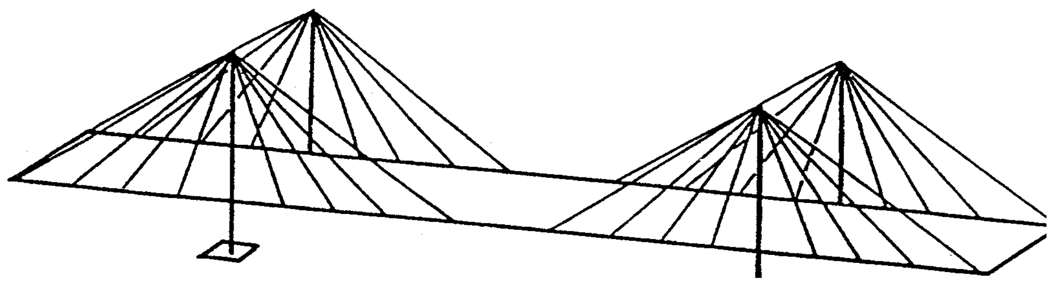


Fig 1.6: Two Vertical Plane Systems

1.7.3.3 Two Inclined Plane System

In this system, two planes of cable-stay are placed on each side of the bridge deck, meeting at a point above the centerline of the bridge. The planes are of sloping type as shown in the fig. The type of pylon suitable for this system is an A-Shaped tower.

This system give rise to the vertical clearance problems for vehicular traffic, therefore require increased tower height or extension of the width of the deck. Joining all cables on the top of this tower has a favorable effect regarding wind oscillations, because it helps to prevent the dangerous torsional movement of the deck. However, the construction of A-Shaped tower is somewhat difficult compared to H-Shaped tower.

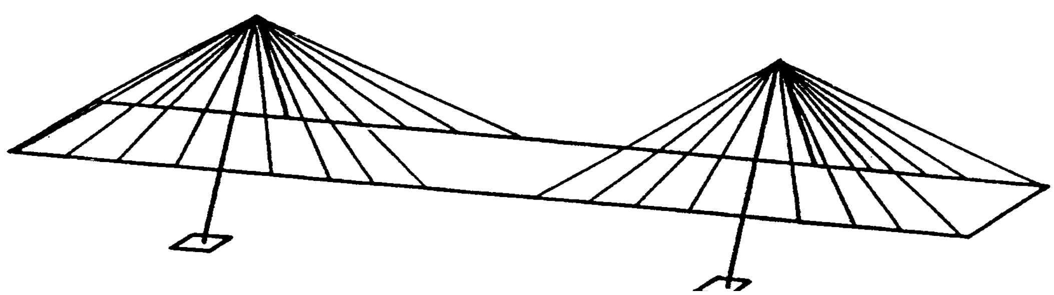
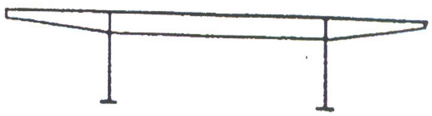


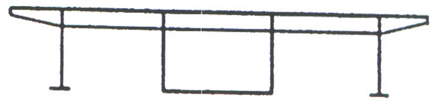
Fig 1.7 Two Inclined Plane

1.7.4 Deck Systems

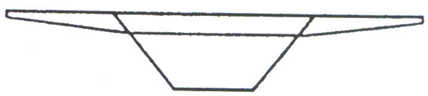
The deck girder is designed to be as slender as possible without compromising its safety against geometric stability. Other design consideration includes the aerodynamic behavior and ease of cable anchorage. The shape of the cross section depends on the type of cable system used. Fig.1.8, Shows various types of deck girders being used in the cable-stayed bridge.



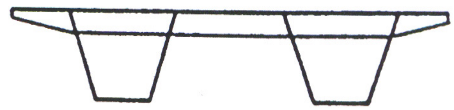
Twin I- Girder



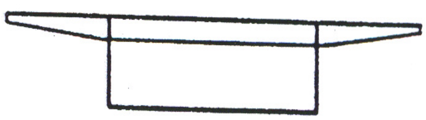
Central Box Girder and Side single Girder



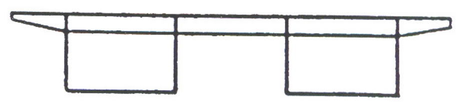
Single Trapezoidal Box Girder



Twin Trapezoidal Box Girder



Single Rectangular Box Girder



Twin Rectangular Box Girder

Fig 1.8 Types of Deck

In the case of two-plane system, hollow or solid slabs with stiffened edges are most economical for deck widths up to 15m. In such systems, the deck does not need to furnish any torsional stiffness, any load that are unsymmetrical with respect to the center-line of the bridge are picked up by the cables, which provide stiff support along the edges. Cable anchors are usually placed outside the deck or within the sidewalks.

In case of axial suspension, box sections must be used to ensure adequate torsional stiffness. Most cable-stayed bridges have orthotropic decks, which differ from one another only as far as the cross sections of the longitudinal ribs and the spacing of the cross-girder are concerned.

The orthotropic deck performs as the top chord of the main girder or trusses. It may be considered as one of the main structural elements, which lead to the successful development of modern cable-stayed bridges. For relatively small spans in the 60-90m range it is convenient to use a reinforced concrete deck acting as a composite section with the steel grid formed by the stringer, floor beams and main girder.

An alternative solution is, reinforced concrete deck acting monolithically with the main reinforced or prestressed concrete girders.

1.7.4.1 Reinforced or Prestressed Concrete Girders

A large number of cable stayed bridges have been built with a reinforced or prestressed concrete deck and main girders. These bridges are economical, possess high stiffness and exhibit relatively small deflection. The damping effect of these monolithic structures is very high and vibrations are relatively small.

1.7.5 Cable Type and Their Properties

The cables used for bridge at present fall in the following categories:

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

1.7.5.1 Parallel-bar Cables

Parallel-bar cables are formed of steel rods or bars, parallel to each other and in metal ducts, kept in position by polyethylene spacers. The bars can slide in the longitudinal direction, which simplifies the process of tensioning them individually. Injection of cement grout, carried out after erection, makes sure that the duct plays its part in resisting the stresses due to the live loads. They are delivered in bars of lengths 15-20 m. Continuity then have to be provided by the use of couplers, which considerably reduced the fatigue strength of the stays.

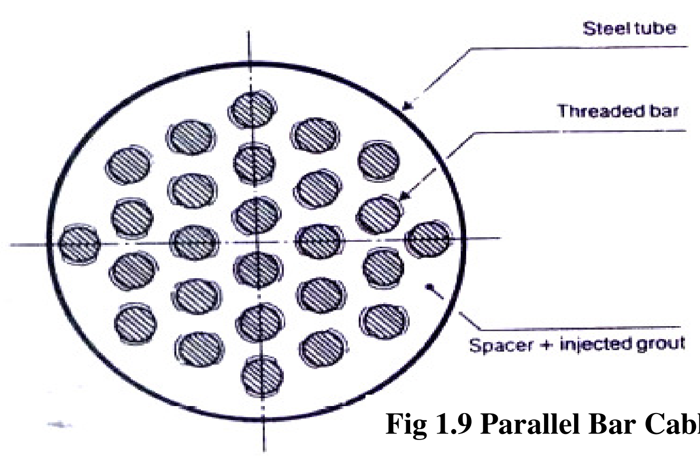


Fig 1.9 Parallel Bar Cable

Capacity of Normal Bars

Quantity of Steel	St 85/105			St 110/125			St 135/15
Nominal Diameter(mm)	26.5	32	36	26.5	32	36	16
Nominal C/S of Steel (mm2)	551.5	804.2	1017.9	551.5	804.2	1017.9	201.1
0.2% proof Stress(N/mm2)	835	835	835	1080	1080	1080	1325
Ultimate Strength(N/mm2)	1030	1030	1030	1230	1230	1230	1470
Ultimate load per bar(kN)	568	828.4	1048.4	678.4	989.2	1252	295.5
Service load per bar (kN)	225.6	372.7	471.8	305.3	445.1	563.4	133

Table 1.2: Capacity of Normal Bars

The use of relatively mild steel necessitates larger sections than when using high-strength wires or strands. This leads to a reduction in the stress variations and thus lessens the risk of fatigue failure.

1.7.5.2 Parallel-Wire Cables

They are high-strength, drawn steel and are placed in metal or polyethylene ducts. The ducts are generally injected with a cement grout after erection. Each stay consists of a bundle of 7 mm dia. Wires, the number varying between and 50 and 350. The stays are thus formed from members which can be delivered on reels, and in operation they can withstand forces of 1300-9000 KN. Their fatigue strength is satisfactory because of their good mechanical properties. They can withstand a stress variation of 350-400 N/mm^2 over two million cycles with a maximum stress of 750 N/mm^2, which is 44% of the tensile strength of the steel.

Capacity of normal Parallel wire cables, 7 mm dia.

Number of Wires	1	61	91	121	163	211	253	313
Nominal c/s of steel(mm^2)	38.5	2348.5	3503.5	4658	6275	8123	9740	12050
0.2%proof stress(N/mm^2)	1520	1520	1520	1520	1520	1520	1520	1520
Ultimate Strength(N/mm2)	1670	1670	1670	1370	1670	1670	1670	1670
Ultimate load (kN)	64.3	3922	5850.8	7779	10480	13566	16266	20124
Service load (kN)	18.9	1764.9	2632.9	3500	4716	6104	7320	9056

Table 1.3: Capacity of normal Parallel wire Cables

1.7.5.3 Stranded Cables

Each strand consists of seven twisted wires, with an external diameter of 12.7 mm or 17.78 mm. When the cables are stressed the lateral stresses which are produced have a bad effect on their fatigue resistance. Also their sensitivity to corrosion is increased since for a given cross-sectional area, the perimeter of the section increases. However the progress associated with effective protection measures has made it possible to overcome these problems in a satisfactory manner

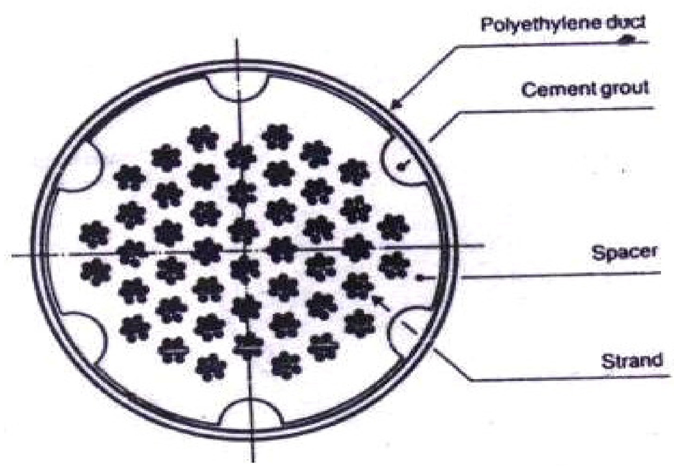


Fig 1.10 Stranded Cable

Capacity of Usual Strand Cables

	37 Strands		61 Strands		91 Strands	
	Ultimate Load	**Service Load**	**Ultimate Load**	**Service Load**	**Ultimate Load**	**Service Load**
Nominal Diameter (mm)						
12.7	6734	3030.3	11102	4995.9	16562	7452.9
15.2	9634	4335.7	15884.4	7148	23696.4	10663.4
15.7	9823	4420.6	16195.5	7288	24160.5	10872.2
17.8	12772	5747.6	21057.2	9475.7	31413.2	14135

Table 1.4: Capacity of Usual Strand Cables

1.7.5.4 Locked Coil Cables

The wires are arranged in successive layers, wound around a central core which consists of circular parallel wires. On the outside, elongated S-section is used and in view of the extant of their overlapping, these form an envelope which is more or less watertight. This effect is even more emphasized by the action of lateral stresses produced during tensioning. Units are obtained which consist of eight or nine layers of 4.7 mm dia. Wires reaching an ultimate load of 6000-12900 KN.

The advantages of locked-coil cables lie in the ease of placing, the economy arising from the fact that ducts and grouting are unnecessary, the reduced anchorage space and the great flexibility which makes it possible to use guiding saddles at the pylons instead of intermediate anchorages.

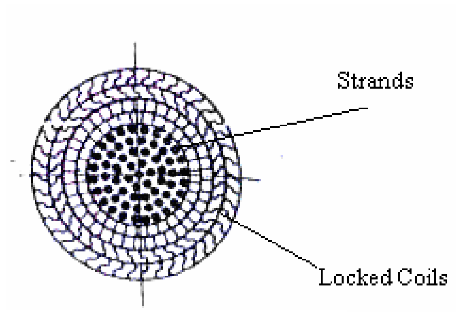


Fig 1.11 Locked Coil Cables

Capacity of usual locked-coil cables

Cable diameter(mm)	**75**	**80**	**85**	**90**	**95**	**100**	**105**	**110**
Nominal C/S(mm^2)	3821	4348	4908	5502	6131	6793	7489	8220
Ultimate Strength(N/mm^2)	1570	1570	1570	1570	1570	1570	1570	1570
Ultimate load(kN)	5999	6826	7706	8636	9626	10665	11758	12905
Service load (kN)	2700	3072	3468	3887	4332	4799	5291	5807

Table 1.5: Capacity of Usual Locked- Coil Cable

The choice of a particular cable depends upon the need of the mechanical properties, i.e. modulus of elasticity, ultimate tensile strength, fatigue, durability etc. and the structural and economic criteria i.e. erection, design anchorages etc. The combination of 50-350 wires placed in parallel position in the case of parallel-wire results into higher modulus of elasticity.

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

The corrosion resistance of locked coil strand is increased by virtue of exterior tightly locked Z or S shaped wires. The modulus of elasticity is high.

The advantages of locked-coil cables in the case of placing are:

→ The economy arising from the fact that ducts and grouting are not necessary.
→ The reduced anchorage space.
→ The great flexibility, which makes it possible to use guiding, saddles at the pylons instead of intermediate anchorage.

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

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

1.8 ANCHORAGE

Where a structure is prestressed, it is possible to place the anchorage sufficiently far from the critical zones to provide that because of the bond produced by grouting, they are subjected to only a small level of stress variation.

In a cable-stayed bridge, the stay act as vertical supports and thus take up directly and completely all of the loads acting on the structure. The anchorages of the stays are thus of vital importance. Researches have produced high performance devices, particularly against fatigue, while still meeting the requirements for minimum spacing. The anchorages must be designed in such a way that a replacement is possible without it being necessary to provide special fittings.

1.8.1 Anchorage for parallel-bar cables

This anchorage system for parallel-bar cables is designed so as to provide the anchorages for the stays with higher fatigue strength than that of the use of a nut on threads on the bar itself. The cable is formed of bars placed inside a steel tube. This tube is extended into the anchorage zone and its bond with concrete along the embedded length is improved by means of rivets. The continuity of this rigid tube is broken during erection, which allows the force due to dead weight to be transmitted directly to the anchorage plates of the bars. Continuity is then re-established and the connection between the tube and the bars ensured by injection of a cement grout. When this connection has been made, the live loads act on the whole monolithic structure and are transmitted to the concrete by bond along the immersed part of the tube. This procedure, however, does have one major disadvantage in that replacement of the stay is well impossible.

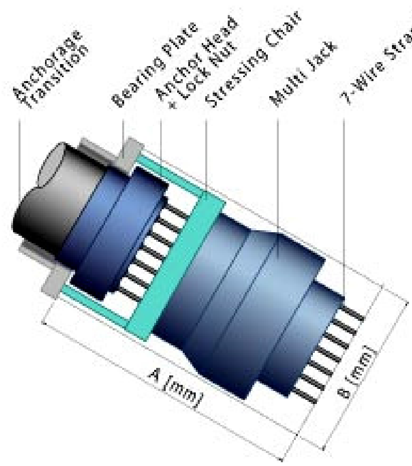


Fig 1.12 Anchorage for Parallel Bar Cable

1.8.2 Anchorage for Parallel-wire cables

The prestressing system consists of parallel wires held by anchorages formed of male and female cones, in heavily reinforced concrete. This economical method proved to be inadequate to meet the demand for members with ever increasing capacities, due to the excessive space required and the difficulty of ensuring locking-off without a slight release.

These problems were overcome by putting in operation an anchorage based on the principle of cold formed upset at the end of each wire.

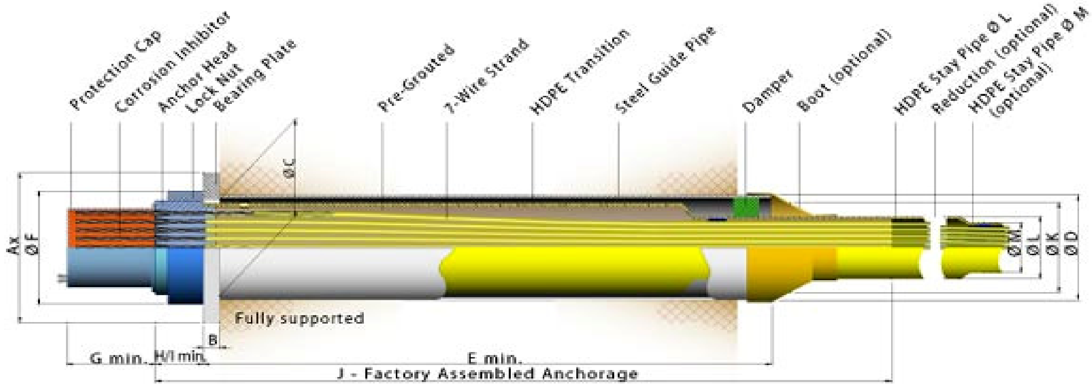


Fig 1.13 Anchorage for parallel wire

1.8.3 Strand Cable Anchorages

The technique of anchoring strand cable, currently used in prestressed concrete, consists of locking the strands using wedges. Although for a given capacity of load, the number of strands required is much less than the number of parallel wires, the placing of wedges near to each other means that the anchorage takes up a great deal of room. The spacing of the strands and the pinching effect which the wedges apply to the strands both tend to reduce the fatigue strength needing special anchorages.

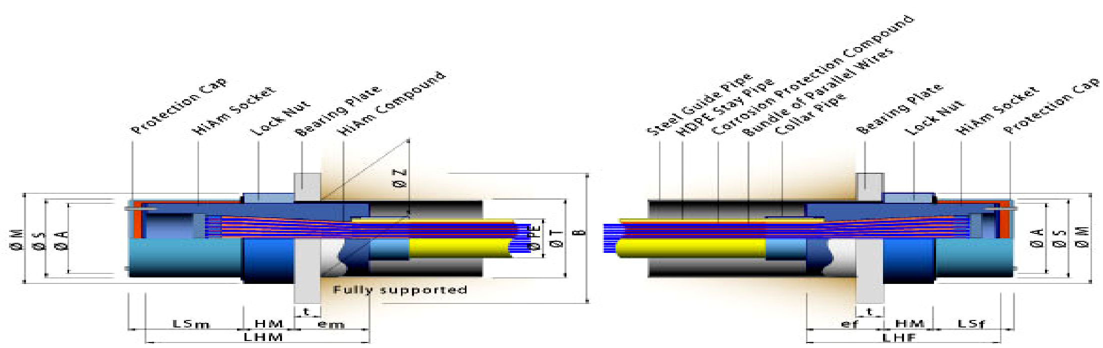


Fig 1.14 Anchorage for Strand Cables

1.8.4 Anchorage for locked-coil cables

The anchorage for a single strand locked coil cable is provided by a massive casing. The cavity, which can be a truncated cone or tulip or other shape, makes it possible to separate the wires which are then immersed in a zinc-based amalgam, poured hot. This method of manufacture results in an appreciable reduction of the fatigue strength of the anchorage, cable reduction of the fatigue strength of toe anchorage which reaches only 120-150 N/mm2 or about 50% of that of the cable itself. However the space whi8ch it requires is relatively small compared with other systems.

1.9 PROTECTION OF CABLES

Stays are structural elements which are very exposed to corrosion and thus appropriate protection measures are needed, these depend on various parameters, notably the type of bridge, its location, loadings and environment, and the importance of the risks arising from corrosion of the cables. It is difficult to judge the suitability and efficiency of these protective measures in view of the limited extent of partial experience available and the gaps in theoretical research on the matter.

1.9.1 Ducts and Tubes

With the exception of locked-coil cables, normal stays are generally fitted with ducts in steel or in a plastic material resistant to a certain extent to the aggressive nature of the environment. The effectiveness of this system depends chiefly on the type of cables used and additional protective measures; in particular injection. The fact that this injection is applied at a late stage can considerably modify the qualities required of the duct.

Ducts in plastic material have flexibility which greatly eases their placing. This flexibility can however give rise to slight undulations along the axis of cables. It is possible to get over this problem by the introduction of a sufficient number of spacers and by limiting the pressure of the cement grout injected.

Injection is sometimes considered to be the weak point of stays. In fact, the variations in loading cause deformations in the cables which can reach 0.5-1.0 mm per meter run.

1.9.2 Galvanizing

Protection by galvanizing has already been proved in several fields of use of structural steel. It is done by steeping or immersing the wires in a bath of melted zinc, automatically controlled to avoid overheating. A wire is described as ‘terminally galvanized’ or ‘galvanized redrawn’ depending on whether the operation has taken place after drawing or in between two wire-drawings prior to wire being brought to the required diameter. For reinforcing bars and cables the first method is generally adopted.

1.9.3 Coating

The coating process, used currently for locked-coil cables, consists of coating the bare wires with an anti-corrosion product with a good bond and long service life. The various substances used generally have high dropping points so as not to run back towards the lower anchorages. They are usually high viscosity resins or oil-based greases, paraffin or chemical compounds. Although this type of protection seems less effective than the others, considerations of economy and manufacture can lead to its use.

1.10 PERFORMANCE OF THE CABLE SYSTEM

The structural analysis of the cable stayed bridge system is based on the assumption that the cable possesses reasonable reserves in tension, which should be greater than the possible compressive forces which may originate at certain positions of the loading. This assumption permits cables to be considered as rigid bars, stressed by tension and compression and providing the geometrical stability of the cable system under arbitrary loading. In cable-stayed bridges with a radial or harp cable system, the stiffening girder performs as a continuous at the towers and at the end anchorage.

The rigidity of the tower and the way it is fixed to the pier or to the stiffening girder do not practically influence the behavior of the bridge system under any loading position. This may be explained by relatively great height of the tower, which means that its deformation with a rigid anchorage is similar to that with hinged support, and part of horizontal forces acting on the tower may be neglected.

When the cables are arranged parallel to each other, the forces from one connection to other can transfer only by bending of the section of the tower between them. This may cause substantial bending moments in the towers. For the reason, with the harp system it is more convenient to attach only one of the cables, and to connect the remaining ones by using movable bearings. The most effective scheme may be to use the stiffening girder.

1.11 CONSTRUCTION METHODS

The various methods of construction of cable-stayed bridges are broadly classified as:

- Staging Method
- Push out Method
- Free Cantilever method with Progressive Placing
- Balanced Cantilever Method

The choice of the construction technique has significant influence on the cost of the project, the final profile of the bridge deck and on the distribution of stresses in the structure. The cantilever method is the most versatile technique and is commonly used in the construction of cable-stayed bridges.

1.11.1 Staging Method

This method is appropriate when the pylon is not designed with full end fixity to the pier or cannot be temporarily fixed, i.e. the pylon is not stable unless the anchor cable is held in position. The figure below illustrates a typical erection procedure beginning at one of the abutments. Temporary piers are first installed and the deck units progressively placed one-by-one and welded together to form short free cantilevers. A derrick-type crane mounted on rail track is commonly used for lifting and thus the weight of a unit would normally have to be significantly less than the derrick capacity (typically about 150 tones at minimum radius), and it may sometimes even be necessary for assembly to be carried out in sections. Prefabrication normally takes place off site, and units are erected in 5-15 m lengths. The length of free cantilever possible during the construction phase depends on the deck characteristics and must be carefully determined for the temporary conditions but over 50 m of unpropped section have been successfully achieved. similar procedure using precast concrete could be used but because of the much heavier weights involved, either shorter sections or specialized lifting carriages would be necessary until the stays were in position.

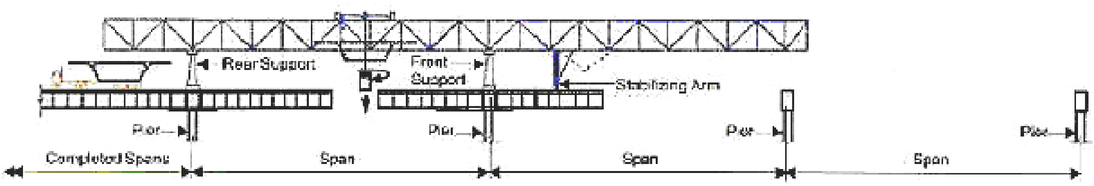
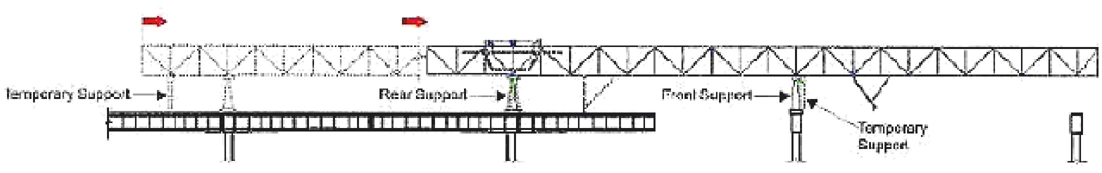


Fig 1.15 Staging Method



On completion of the deck, all the stays are connected tensioned and the temporary piers dismantled. However, some extension of the cable is unavoidable as the self-weight of the deck is taken up. The temporary propping should therefore he erected at a height calculated to allow for this movement.

1.11.2 Push out Method

The push out method is used in those cases where case must be taken not to interfere with the traffic below the bridge. In this method, the large section of deck is pushed out over the piers on roller bearings. The deck is pushed out either from both abutments towards the center or in some instances from one abutment all the way to other abutment.

The components are fabricated or cast, put together and then gradually rolled out over a span in a process somewhat similar to mass production. Thus it ensures the quality and low cost.

1.11.3 Free Cantilever Method with Progressive Placing

In many situations the installation of temporary supports would be difficult and expensive and cantilever construction might be considered as an alternative. Figures below show a typical example whereby the side spans are constructed on temporary propping followed by the tower. This part of the bridge is often situated on the embankments where access may favor the use of cranes at ground level.

The centre span is thereafter erected unit-by-unit working out as a free cantilever from the tower or pylon. Like in the previous method, steel box sections up to 20 in long are commonly lifted either by derrick or with mobile lifting beams and welded into place. Thereafter the permanent stays are fixed each side of the tower and the bending moment caused by the cantilevering section removed.

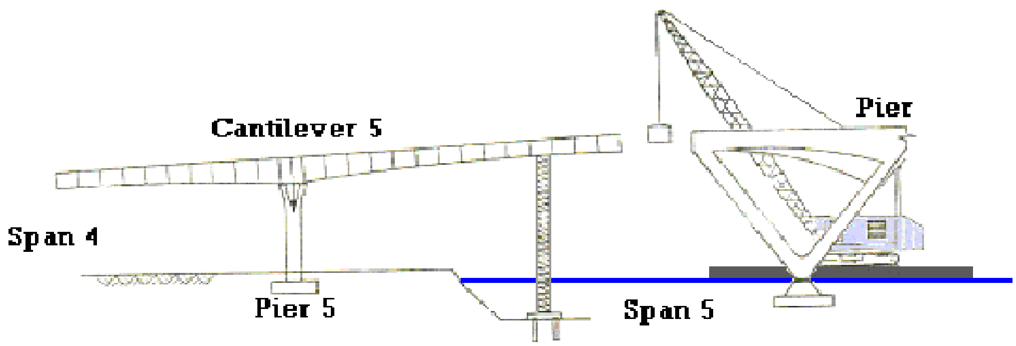


Fig 1.16 Free Cantilever Method

The provision of temporary stays is particularly important with precast concrete segments where units weighing up to 300 tones are occasionally erected. The normal procedure is to match cast adjacent segments an subsequently glue the joints with epoxy resin, temporary post-tensioning being applied to bring the two elements together. The permanent cab is tensioned simultaneously as the temporary stay is released.

An in situ concrete cable-stayed deck constructed with a mobile carriage and formwork similar to that used in cantilever construction an alternative to steel and precast concrete, but a rate of progress of or 3-4 in section each week is very slow and thus is more commonly adopted as an alternative to stepping formwork systems on multispan bridges in the range 30-70 m between piers.

The cable-staying technique using temporary stays only has also proved successful for multi short span bridges of the precast type. This progressive erection method allows units to be transported along the previously constructed deck, which are then swung round and attached to lifting equipment such as swivel arm. The stays are usually tensioned with built-in hydraulic jacks, and the whole device moved forward from pier to pier as each span is erected and post-tensioned.

1.11.4 Balanced Cantilever Method

The occasional need to have clear uninterrupted space below the bridge, for example railway sidings, private property, etc., has forced designers and constructors to develop the balanced cantilevering technique, whereby all or at least very few props are required, as shown in figure below. Erection proceeds simultaneously each side to the tower, with the first few sections over the piers, temporarily supported on false work until the tower has been erected and the cables attached. Like the other methods, a degree of cantilevering beyond the last attached cable may be possible depending upon the capability of the section to resist bending movement, the potential for this possibility being much better for steel plate than heavy precast concrete segments.

An important feature of this technique is the need to have a stiff tower and fixity between the deck and tower and its foundations, because of imbalances caused by construction plant, variation in segment dead weight, and tension in the cables. Where possible, the tower design should be selected to accommodate this requirement, otherwise substantial extra staying, tempo anchor cables or a heavy deck tower fixing clamp must be provided. Cantilever ever spans over 150 m each side of the tower are commonly erected, but where ever possible some propping is desirable to aid stability.

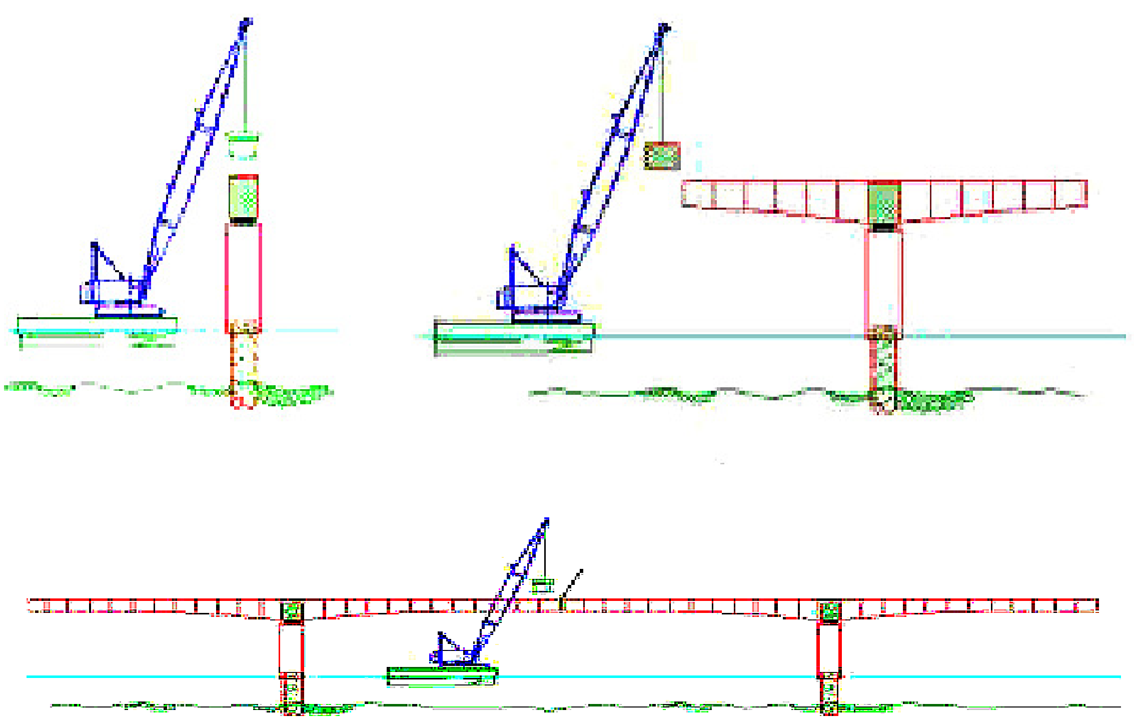


Fig 1.17 Balanced Cantilever Method

1.12 COMPARISION OF CABLE STAYED BRIDGES WITH OTHER BRIDGES

The cable-stayed bridges are superior to the other bridges because of the following points:

1) The deflection of cable-stayed bridges is very small hence it is much stiffer.
2) The quantity of steel required for erection and construction is less.
3) It does not require large or heavy anchorage for the cables.
4) The use of correct analysis of the structural system.
5) The use of tension members having under dead load a considerable degree of stiffness due to high pre-stress and beyond this still sufficient capacity to accommodate the live load.
6) The use of erection methods and methods to ensure that the design assumptions are realized in an economic manner.
7) The development of methods of structural analysis of highly statically indeterminate structures and application of electronic computers.
8) The development of orthotropic steel decks.
9) Application of high strength steels, new methods of fabrication and erection.

1.13 CABLE STAYED BRIDGE Vs SUSPENSION BRIDGES

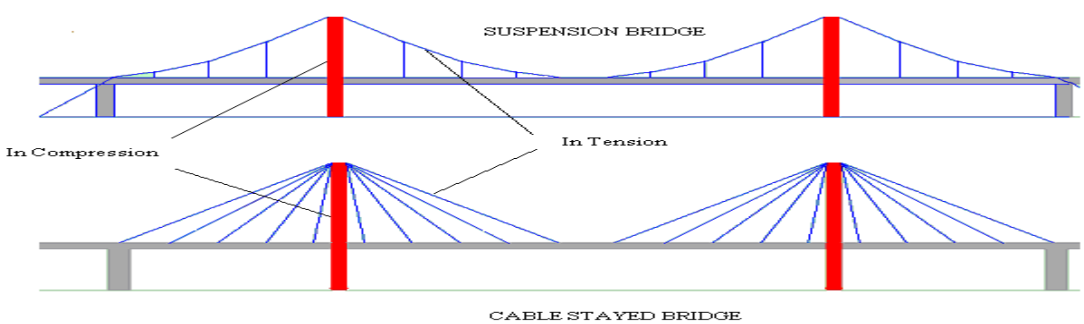


Fig 1.18 Nature of Forces

No.	Suspension Bridge	Cable-Stayed Bridge
1.	It is an Determinate Structure	It is Highly Indeterminate structure
2.	The deck of a suspension bridge merely hangs from the suspenders, and has only to resist bending and torsion caused by live loads and aerodynamic	The cable-stayed deck is in compression, pulled towards the towers, and has to be stiff

	forces.	at all stages of construction and use.
3.	A suspension bridge has terminal piers, unless the ends are joined directly to the banks of the river. The cables often pass over these piers and then down into the ground, where they are anchored, and so the piers have to redirect the tension.	The cable-stayed bridge is well-balanced, the terminal piers have little to do for the bridge except hold the ends in place and balance the live loads, which may be upward or downward, depending on the positions of the loads
4.	The deck is supported from loosely hung main cables through vertical hanger. This results into a flexible structure.	The deck is supported directly from the towers with inclined stay-cables.

Table 1.6: Comparison of Cable Stayed and Suspension Bridge

1.14 SOME IMPORTANT CABLE STAYED BRIDGES ALL OVER THE WORLD

Sr. No.	Bridge	Country	Span	Year
1.	Tatara	Japan	890 m	1999
2.	Pont de Nrmandie	France	856 m	1995
3.	Second nanjing	China	628 m	2001
4.	Third wuhan	China	618 m	2000
5.	Qingzhou Minjiang	China	605 m	2001
6.	Yangpu	China	602 m	1993
7.	Xupu	China	590 m	1997
8.	Meiko Central	Japan	590 m	1998
9.	Rion-Antirion	Greece	560 m	2004
10.	Skarnsunder	Norway	530 m	1991
11.	Queshi	China	518 m	1999
12.	Tsurumi Tsubasa	Japan	510 m	1994
13.	Jingsha	China	500 m	2002
14.	Ikuchi	Japan	490 m	1991

15.	Oresund	Sweden	490 m	2000
16.	Higashi-Kobe	Japan	485 m	1992
17.	Ting Kau	China	475 m	1998
18.	Seo- Hae	South Korea	470 m	2000
19.	Ales-Fraser (Annacis)	Canada	465 m	1986
20.	Yokohama Bay	Japan	460 m	1989
21.	Second Hoohgly	India	457 m	1992
22.	Second Severn	Britain	456 m	1996
23.	Surgut	Russia	408 m	2000
24	Karnali	Nepal	325 m	1993

Table 1.7: Important Cable Stayed Bridges of World

CHAPTER 2
METHODS OF ANALYSIS

A number of techniques can be used for the analysis of cable-stayed bridges. Examples include the use of a scaled-down model for testing, and the use of an analytical model which considers the linear and nonlinear behavior of the cable-stayed bridge when subjected to static and dynamic conditions of load. For the dynamic model, certain parameters should be defined and idealized, such as the restraints at the joints, the stiffness or flexibility of each member, and connections between the cables, stiffening girders and towers. One- and two- planes systems of the bridge may be considered as one and two dimensional space frames respectively.

2.1 LINEAR ANALYSIS AND PRELIMINARY DESIGN

Cable stayed bridge systems are generally many times statically indeterminate. A statically determinate basic system may be formed by different methods. The deflections of the basic system under applied loads may be determined by applying the classical theory of structures or so-called first order theory, by neglecting the deformation of the system when formulating the equilibrium conditions.

For a statically determined basic system, the resulting equations are linear in the loads and in the internal forces, and linear superposition is valid for the internal forces caused by different loads or load groups.

If Hook's law is assumed to be valid, linear superposition applies also to the displacements, and therefore to the determination of the stresses of cable-stayed bridge systems.

If the analysis of cable stayed bridges is based generally on the assumption that the elastic displacements of a structure are proportional to the applied load, it is defined as linear behavior. However, for the case of cable stayed bridges this assumption has been proven to be approximate and for large spans unsafe. When the actual performance of the bridge is analyzed and the final deformed geometry is considered, then the loading moments, deflections and stresses have larger magnitudes if nonlinearity is neglected.

2.1.1 Preliminary design

The design process fro a cable stayed bridge system with accepted geometrical layout may be divided into the following stages:

A preliminary set of sectional properties is assumed for each member of the system.

The sectional properties assumed in stage are analyzed applying one of the statical methods of analysis. Stresses and displacements under the given loads on the system are determined and compared with the maximum unit stresses and maximum displacement span ratios allowed by the specifications.

A new set of sectional properties is chosen to satisfy the requirements of the specifications. The above stages are repeated until we obtain a specified relation between the sectional properties assumed in stage and those obtained in stage one and those obtained in stage two.

Using a classical method, we have to carry out a structural analysis of a cable stayed bridge system by manual computation. One of the disadvantages is that we must assume linear behavior of our bridge system. This is because it is highly impractical to attempt a manual iteration of a system of equations associated with nonlinear behavior.

2.2 NONLINEAR ANALYSIS

Nonlinear performance of the cable stayed bridges generally depends on the behavior of the cables, stiffening girder and pylons.

2.2.1 Nonlinearities of Cables

Nonlinearity of the cables originates with an increase in the loading followed by a decrease in the cable sag, which produces an elongation of the cable and corresponding axial tension. To overcome this nonlinear effect, the method of equivalent modulus of elasticity was proposed to include the normal modulus and the effect of sag and tension load. These factors are expressing changeable stiffness of stay cables. Actually, the stiffness depends on the tensile stress, length of the cable and its deflection.

The equivalent or ideal modulus of elasticity of the cable as expressed by Ernst is:

$$\mathbf{E_i = E/ 1+ (\gamma^2 l^2 E/12\, \delta^3)}$$

E= Modulus of elasticity of straight cable.

L = Horizontal length of the cable.

γ = Specific weight of the cable

δ = tensile stress in the cable

2.2.2 Nonlinearity of stiffened girders and pylons

When stiffened girders and pylons are subjected to the simultaneous action of compression loads and bending moments, then the interaction of loading and axial forces results in nonlinear behavior. The degree of nonlinearity depends on the intensity of the compressive load compared with the buckling load and the magnitude of deflection caused by the bending action. Because of the presence of high compressive forces in the relatively slender stiffening girder and towers, the girder and the towers need to be analyzed as a beam column. The axial compression force increases the bending moment of the beam column, and the resulting relationship is nonlinear.

2.2.3 Nonlinearity due to deformation of the structure

In a cable stayed bridge, the deformation of the superstructure under loading affects the value of the stresses. Therefore the principle of superposition may be applied only with certain limitations. This problem is treated by the deformation theory or so called second order theory buy taking into account the effect of the deflections of the structure in calculating the stresses and forces. The internal forces of the bridge are not directly proportional but grow at a faster rate than the external loading. Due to the deformations of the structure there are additional stresses, which are not proportional to the additional loads. These stresses may be determined by the method of successive approximation. At the first stage, the stresses are calculated considering the initial geometry of the structure, applying the principles or linear analysis. The deformations obtained are used further to determine the modified geometry of the structure. At the second stage, the linear analysis is applied again for the structure with modified geometry. This method is repeated until the deformations remain constant from one stage to the next.

2.3 DYNAMIC ANALYSIS

The role of dynamic forces in cable stayed bridges is very important. More than for nay other type bridge, such forces can even determine the very feasibility of the project. There are, in general three types of problems:

- Aerodynamic stability
- Physiological effects
- Safety against earthquakes

The aerodynamic behavior of a cable stayed bridge determines, to a great extent, the safety. It is a fact that lack of overall dynamic stability was the reason for the collapse of a number of the earliest suspension bridges.

Without damaging the structure, the vibrations due to wind and traffic can cause inconvenience to users. These physiological effects are generally very subjective experiences.

Analysis of all these dynamic phenomena, calls for prior knowledge of the frequencies and vibration modes of the structure concerned.

2.4 NATURAL FREQUENCIES AND PRINCIPAL MODES OF VIBRATION

2.4.1 Methods of determination of natural frequencies

There are a number of methods for calculating the frequency of a structure.

To simplify the numerical operations, the mass of the structure is concentrated at a certain number of separate points and the influence of damping is neglected. The methods treated are the following:

- Classical method based on the differential equation of movement: method of determinants; method of matrix iteration
- Rayleigh's method based on considerations of energy: simple method; iterative method.

2.4.2 Classical Method

The equation of movement of an undamped system with n degrees of freedom, in free oscillation, is written in matrix form

$$\mathbf{MV + KV = 0}$$

When the system oscillates in the vibration mode shape at level j, all the masses carry out, in synchronization, harmonic motions with the same period and the same phase. As a result:

$$\mathbf{V = Vj \sin (\omega jt + \varphi j)}$$

An expression in which Vj characterizes the deformation of mode j, ωj the circular frequency, φj the phase and K the matrix of stiffnesses. Combining the two equations above gives

$$\mathbf{-\omega^2 j\ M\ Vj \sin(\omega jt + \varphi j) + K\ Vj \sin (\omega jt + \varphi j)= 0}$$

As this relationship has to be verified at any moment, it can be simplified by dividing by sin(ωjt +φj):

$$\mathbf{(K- \omega^2 jM)\ Vj = 0}$$

Mulitplying this equation by the ratio $F/ \omega^2 j$, F being the flexibility matrix, gives

$$\mathbf{(F/ \omega^2 j)\ (K- \omega^2 jM)\ Vj = (F/ \omega^2 j)O}$$

$$\mathbf{((FK/ \omega^2 j)- FM)\ Vj = O}$$

And by using the well known relationship FK=I;

$$\mathbf{((I/ \omega^2 j)- FM)Vj = O}$$

In order that there shall be solutions to equations which differ from Vj=O, the determinant of the unknown coefficient must be zero, i.e.

$$\mathbf{Det\ |K- \omega^2 jM| = 0}$$

$$\mathbf{Det\ |I/ \omega^2 j) - FM| =0}$$

These two equations are known as the characteristic equations of the system, the first established on the basis of the matrix of the stiffnesses of the second on the basis of the matrix of flexibilities.

Development of one or other of the determinants leads to an algebraic equation of degree n in ω^2, and the n roots of this equation (ω^2, $\omega^2{}_{2,}$ $\omega^2{}_{3,}$ …. .., $\omega^2{}_n$) characterize the frequencies of n modes of vibration.

I we replace ω^2 by its value in equations; we get a matrix equation of order n in Vj. A given value is attributed arbitrarily to one of the terms of the vector and the deformation of the mode examined is found by solving the matrix equation. The search of self-frequencies and modes by the solution of the characteristic equation becomes tedious for systems with more than three or four degree of freedom.

2.4.3 Method of matrix iteration (Stodola)

Instead of seeking an exact solution of the characteristic equations, an iterative process is followed from an initial value of Vj, arbitrarily chosen. If the equation is used, the method converges fairly rapidly towards the fundamental form and thus enables the development of the determinant to be avoided.

2.5 RAYLEIGH'S METHOD

2.2.1 Simple Method:

Rayleigh's method, which is based on the principle of conversation of energy, is very useful in the approximate study of a system with a large number of degrees of freedom. In the case of an undamped free vibration, ω is obtained from the conservation of kinetic and potential energies. The accuracy of the method depends on the valuation placed on the deformation, which should as soon as possible, approach one of the vibration mode shapes. In general, the work is limited to a search for the fundamental frequency.

A reasonable deformation Vj for the system is assumed generally obtained by the application of any static loading case Pj on the masses m_j.

The maximum kinetic and potential energies, Ec and Ep, of the masses thus displaced, are equated.

In the case of a system with one degree of freedom, the application gives;

$$E_p = pv\ (t)/2$$

$$Ec = mv^2\ (t)/2 = (mv^2/2)\ \omega^2 \cos^2 \omega t$$

$$E_{\,p,max} = pv/2 = E_{c,max} = (mv^2/2)\ \omega^2$$

Hence,

$$\omega^2 = (pv/mv^2)$$

The expression may be generalized, for a system with n degree of freedom,

$$\omega^2 = \sum\nolimits^n{}_{i=1} p_{i\ vi} \,/\, \sum\nolimits^n{}_{i=1} m_i\ v_i^2$$

Concerning the choice of the load Pj, it is usual to take as the reasonable deformation of the system that given by its masses mi acting like static loads Pi = gmi, in the direction of the required mode. The above equation becomes;

$$\omega^2 = g \sum^n{}_{i=1} m_i v_i / \sum^n{}_{i=1} m_i v_i^2$$

It is possible to limit the numerical calculations by choosing for Pj a single load Pi on the mass mi with maximum displacement. If this is taken as unity (pi=1), we get

$$\omega^2 = v_{max} / \sum m_i v_i^2$$

2.5.2 Iterative Method

To improve the accuracy of the method given above, an iterative process can be adopted. Proceeding from step (a) to step (a+1) then gives

$$P_i^{(a+1)} = gm_i \, v_i^{(a)} / v_n^{(a)}$$

$$\omega 2 = \sum^n p_i^{(a+1)} v_i^{(a+1)} / \sum^n{}_{i=1} m_i (v_i^{(a+1)})^2$$

The process can be repeated indefinitely, but convergence is fairly rapid, so that two or three repetitions are generally enough to obtain a satisfactory result.

2.6 NONLINEAR SOLUTION PROCEDURES

There are three basic techniques for solving the nonlinear equation, when the stiffness varies as a function of nodal displacements and member forces.

2.6.1 Incremental or Step by Step Method

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

This procedure is simple to apply and has been widely used, since no iterations are required during the whole analysis. The disadvantage of this method is that errors are likely to accumulate after several steps unless very fine steps are used.

2.6.2 Iterative or Newton Methods

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

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

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

2.6.3 Step-Iterative or Mixed Method

In the Mixed method, the load is applied incrementally, but after each increment, successive iterations are performed using either Newton-Raphson or Modified Newton-Raphson iteration within each load step. In this mixed method, the unbalanced joint loads reapplied incrementally during each cycle of iteration. The stiffness matrix of the structure is recomputed at the beginning of each load increment. At the end of each iteration cycle, the unbalanced loads at joints are recomputed and applied incrementally during the next cycle.

Both mixed methods yield high accuracy at the price of more computational efforts. Either of them must be used in the analysis of cable-stayed bridges to avoid divergence of the solution, if the total load is applied in one step.

2.7 COMPUTER USE FOR ANALYSIS

Except in the case of a very simple cable stayed bridge, a computer is necessary for the solution of this type of structure, being used primarily for analysis rather that for a design application. Computer programs are necessary to generate the influence diagrams of cable forces, stiffening girder, bending moments and shears, tower and pier reactions. The computer is also required for rapid evaluation of effects and loadings of various parameters that have been taken into account in achieving a reasonably efficient design. Probably the most important problems is the determination of the optimum section of the stiffening girder section, and cable configuration and size. The nonlinear behavior of cables, whose sag varies with changing axial load, presents problems in the solution of the bridge system more complex than those of the structures of linear behavior. A convenient method of accounting for the nonlinear behavior of the stay cable bridge system is to introduce the concept of a straight line chord members with a modified or ideal modulus of elasticity substituted for the actual cable members. The use of this concept allows the application of a plane frame computer program properly adapted to account the nonlinearity by an interaction procedure.

2.8 SEISMIC BEHAVIOR

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 ad 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.

2.9 SEISMIC ANALYSIS

The design of bridges subjected to seismic excitation can be carried out in two ways. Either an equivalent static analysis is prepared, using a carefully selected substitute load or a complete dynamic analysis is carried out.

In the first case, the expression for the substitute static load equivalent to the maximum dynamic effect of the earthquake is always of the form:

$$H = \lambda G$$

An expression in which λ represents a fraction of the permanent loads, g varying from .05 to 0.2. This force H, assumed to act horizontally is distributed over the height of the structure in accordance with a dynamic law. Regulations dealing with earthquake design define H and the distribution law. Seismic forces calculated in accordance with this method are well below actual forces, in particular where there are earth tremors of high intensity. This can be explained by the fact that deformations beyond the elastic range are tolerated, since the construction of a bridge which will remain in a elastic condition during major earthquakes would be very difficult indeed.

Dynamic analysis generally makes it possible to get a better grasp of the problem, particularly for major structures. It is based directly on solving the equation of movement. It is possible to include the interaction between the pier, the deck and the foundation.

2.10 RESPONSE SPECTRA

The response spectrum is a means of characterizing an earthquake. It is no longer defined by its displacement, speed or acceleration, but by its maximum influence on a simple oscillator. Movements are not measured on the ground itself but in a series of oscillators covering a large range of natural frequencies ω and damping ξ, the response spectra can be distinguished:

S_d, displacement spectrum

S_v, velocity spectrum

S_a, acceleration spectrum

Given that the response of an oscillator to seismic excitation is sinusoidal, the values of the response spectra S_d, S_a and Sv are directly linked to the frequency ω as a divisor or multiplicand:

$$S_a = \omega\, Sv = \omega^2 S_d$$

The form of this relationship makes it possible to represent them on a simple trilogarithmic diagram.

2.11 SPECIFIC PROBLEMS WITH CABLE STAYED BRIDGES

Cable stayed bridges are characterized by the fact that they are very flexible, with natural frequencies of the order of 0.3-1 Hz. As a result this structures are theoretically little sensitive to seismic excitation. However, it is clear that the principal problems lie at the level of horizontal seismic loading. The vertical component can be taken up by the pylons and the stays, the deck being suspended at a multitude of points, which prevents local deformations surpassing the elastic limits.

Codes dealing with seismic design advocate structural and design measures relating to the choice of site, design of foundation, quality of materials of construction, geometric shape and structural details.

Owing to the fact that a certain amount of deformation beyond the elastic range is permissible limit, the bridge must have some capacity for plastic deformation, sufficient to be able to absorb the excess energy transmitted to it by seismic excitation. It is thus necessary to make structural arrangements which will prevent serious damage to the structure during an earthquake of high intensity. The principal aim is to limit differential displacements between the abutments, the piers and the deck; to prevent excessive displacements between the infrastructure and the deck and to limit the risk of collapse of the superstructure.

CHAPTER 3

LITERATURE REVIEW

AGRAWAL T.P AND KRISHNA.P gave in detail the effects of various parameters on the behavior of Cable-Stayed Bridge. The Parameters are (1) Flexural rigidity of towers, (2) Effect of tower height (3) Flexural rigidity of longitudinal rigidity (4) Cable stiffness (5) Central panel length (6) Configuration. The Deflection and force in Harp is greater than fan type. Tower moment increases with increase in stiffness of tower. Girder moments, deflection and cable tension are not affected by change in rigidity of tower. Cable tension increases with increase in stiffness of cables. Force in cable decreases with increase in tower height. For smaller no. Of cables cost of harp arrangement is higher. The effect of cable area on the cost of steel in cables is small. The effect of cable area on the cost of girder and total cost of bridge is large for small values of cable area. The cost of girder is much larger than cable for harp as well as fan type configuration. Cost of girder decreases with the increase in length of cable. Cable tension increases with decrease in central panel length.

PRAKASH SHRESTHA AND AKIO HASEGAWA A method of nonlinear analysis has been established for cable supported structures, which features the separation of cable analysis, and the frame structures with any original configuration and to be conveniently utilized for routine use in practical application. The formulation has been intended to incorporate nonlinear behavior of cable element for finite displacement behavior of the structure. Two-dimensional analysis cannot predict the real behavior of the structure, which is very misleading and gives a very stiff solution. Non-iterative techniques can be applied to the finite displacement analysis of cable-supported structures with enough accuracy.

S.P.SEIF AND W.H.DILGER Both geometric and material Nonlinearity affects the analysis results of prestressed concrete cable-stayed bridges. These effects dramatically reduce the estimated factor of safety of the structure. Material Nonlinearity contributes more to these differences than geometric Nonlinearity. Ignoring the Nonlinearity in the analysis of prestressed concrete cable-stayed bridges results in an underestimation of the forces resisted by the edge cables and in an overestimation of the forces of cables closer to the towers. The Nonlinearity affects the distribution of internal forces in the bridge, and the external reactions significantly. With regards to the strengths of the different cable stayed systems investigated the radiator type of cable arrangement is superior to the other types of stay cable layouts.

P.K.A. YIU ANDD.M.BROTTON Fixed or pinned joint between deck and the tower base is placed. In case of simply supported connection where the deck is simply supported by the tower, two very long axially stiff arbitrary truss members are introduced together with a vertical roller. Where the cables are fixed to the tower the connections are modeled as pinned. The saddle can be modeled as a short axially very stiff truss member, which connects the cable and tower elements. One very long axially very stiff arbitatory element truss element and one very stiff frame elements are introduced connecting the cable and the tower. Cables are truss element while deck as frame element. Deck may be fixed to the pier or allowed to move horizontally on them, fixed or roller support facility may be used. In two-plane system, two girders are connected by cross girders, which are represented by cross member with appropriate properties. A simpler arrangement of modeling of deck is possible, where the main girders and the cross-girders are connected into a single centrally placed girder with equivalent elastic property.

YEW-CHAYE LOO AND GLEN ISEPPI To account for cable sag; Ernst's formula for the effective modulus of elasticity is employed. As Ee is a function of the prevailing cable tension, an iterative process of analysis becomes mandatory. It is found that cable sag has the beneficial effects of reducing the share of the longest cables in supporting the dead load. However, the burden thus relieved is taken over by the adjacent shorter cables, which experience a considerable increase in tension. For full size bridge, the nonlinear effects on the structural response are significant and cannot be ignored in design. The effects are less pronounced in the case where the cables are pretensioned.

TOSHIYUKI NISHIMURA AND TOSHIYUKI KITADA The reasonable effective buckling length can be taken as zero for pylons subjected to pure bending and 0.7 times the height for the pylons subjected to pure compression. The effective buckling length subjected to any combination of axial force and bending moment can be determined by the linear interpolation with these two extreme values. By using effective buckling length, the ultimate strength of pylons can be safely and more accurately predicted than the conventional design method.

G. NARAYANAN, C.S. KRISHNAMOORTHY AND N.RAJAGOPALAN Deflection theory predicts significant increase in longitudinal and transverse moments in tower and deck as compared to elastic theory. The Nonlinearity due to cable sag is more pronounced during erection stages. The Nonlinearity due to beam-column action is of the order of 10%. The Nonlinearity due to large deformations is of the order of 6-12%. With 3D models nonlinear analysis predicts significant increase in out of plane displacements and moments and hence there is a need to study more on the effects of spatial distributions of vehicular load, wind and seismic loads on the axial, flexural and torsional characteristics of cable-stayed bridges. The inelastic analysis indicated possibility of failure either due to tower instability caused by axial forces or by cable rupture.

LIU GUANGDONG AND ZOU YINSHENG It is necessary to derive the resolved formulation of the tangent stiffness matrix for the geometric nonlinear analysis of cable-stayed bridge. The geometric Nonlinearity should be considered in the static and dynamic analysis of long span cable-stayed bridge.

BENJAPON WETHYAVIVORN AND JOHN F. FLEMING In the response spectrum analysis, adequate results can be obtained fro the main span displacements by combining the first five modes. For the tower and side span displacements approximately eighteen modes are required. There is a definite correlation of the earthquake excitations and the responding modes. Vertical excitations results primarily in in-plane symmetric movements. Longitudinal excitation results in anti-symmetric in plane movement. Transverse excitation excites both lateral sway and torsional movements. Out-of-plane multiple support excitation can significantly modify the response of the structure compared to in-phase excitation. The out-of-plane excitation can result in either magnification or reduction of selected responses. The maximum displacements can be accurately predicted by a response for combining the modal responses.

KV RAMANA REDDY, R RAMESH REDDY AND J NANAK RAM Onset of torsional instability for a box girder section is not affected by the presence of a moderate amount of turbulence. With respect to a yawed wind, tie was found that the component of wind normal to the deck governs the torsional instability and buffeting behavior. The buffeting response increases dramatically during erection, due to the decreased frequencies, which emphasizes the need for temporary tie downs. The response to buffeting for a partially erected span is not as large as buffeting theory would predict. It is also evident that the level of structural damping plays only a minor role in the vertical buffeting response at high speeds since the aerodynamic damping dominates.

S V RAJEEVA AND J MOUNESWARA The deck with a high moment of inertia is not favorable, as it increases the bending moments both in the deck and the pylons. The maximum deformation in the deck decreases as the moment of inertia of the deck increases. The decrease in deformation of the deck is only for increase of the inertia of the deck for certain range. After this range the effect of the moment of inertia of the deck on the maximum value of the deck deformation is not significant.

K V RAMANA REDDY, R RAMESH REDDY AND MANOHAR RAO For bridges of the prestressed box girder type the flutter behavior can be predicted reasonably well in advance by calculations by using "thin plate theory". The problem of resonance vibrations due to vortex shedding has adequately been studied by means of model tests in a wind tunnel and found the vibrations amplitudes are almost coinciding with the full-scale values. The bridge showed mainly oscillation in single degree of freedom modes. The oscillation in the bending mode was quite predominant. The torsional mode of oscillation has consistently shown a relatively weaker tendency. From the test it can be concluded that with the basic configuration the full-scale bridge is likely to show flexural oscillations of 50mm, at a wind speed of 30-m/s. The oscillations tended to be larger at positive angles of attack and smaller at negative angles of attack, although the critical speed was only slightly affected.

J A DESAI, B B MISTRY AND J C VYAS The control of the bridge deck profile during the construction of the structure is accomplished by accurate prediction of the displaced geometry of the bridge at each stage. The effect of maintenance or retrofit operations on the state of forces and deformations in the structure can also be assessed. The results of the time-dependent analysis show clearly the significance of time-dependent effects in a multistage structure of this nature where elements of different creep and shrinkage properties are connected. An increase in deflections at 10,000 due to time-dependent effects in bi-stayed system and self-anchored system is about 75% and 133% respectively. Similarly, an increase in cable tensions near the tower is 54% and 110% in bi-stayed and self-anchored systems respectively. At the Center of the Bridge, the increase in cable tension is very small. Live load to dead load ratio is very low in prestressed concrete cable stayed bridges; fatigue effect is not significant in the cable stays. Similar to steel bridges, prestressed concrete bi-stayed system is found to be economical that the self anchored system. A reduction in the total cost of girder and due to omission of intermediate side span supports of bi-stayed system is found to be about 72% more than the additional cost of anchor blocks.

N RAJAGOPALAN The holistic approach of design of cable stayed bridges requires simultaneous decision on configuration of cables, the span ratios of main and anchor or side spans, the shape, type and dimensions of the tower, the type of deck, the anchorages for the cables on the deck, the sequence of erection, etc. Such a configuration will lead to compatible design for construction procedure. The final analysis, design, detailing, construction procedure and monitoring during construction can be incorporated in the design manual. This would definitely lead to aesthetically pleasing, structurally efficient, cable stayed structure, which could be completed with ease of construction with minimum maintenance requirements.

J.M. Caicedo, G. Turan, S.J. Dyke, and L.A. Bergman This paper presents a comparison of two approaches to finite element modeling of the dynamic behavior of a cable-stayed bridge. The bridge considered in this study is the Cape Girardeau Bridge, which has a composite deck composed of two steel girders along the edges spanned by a concrete slab. The first model of the deck assumes of a rectangular cross section with lumped masses at the deck level, while the second model includes additional lumped masses used to model the tensional behavior of the deck. Modal characteristics of the two models are compared to examine the effects of neglecting the shape of the cross section. The second model is shown to offer improved torsion responses, because the locations of the mass center and shear center are correctly accounted for in the model.

Raid Karoumi The response of a cable-stayed bridge under a moving vehicle is complex due to the interaction between the bridge and the vehicle. This paper describes a method of evaluating this response by idealizing the bridge as a Bernoulli-Euler beam on elastic supports with varying support stiffness. The analysis uses the mode superposition technique and calculates the response in time domain, utilizing an iterative scheme. To illustrate the efficiency of the solution methodology and to highlight the dynamic effects, a numerical example is presented.

E.A. Johnson R.E. Christenson & B.F. Spencer Jr. Cables, such as are used in cable-stayed bridges and other cable structures, are prone to vibration due to their low inherent damping characteristics. Transversely-attached passive viscous dampers have been implemented on some cables to dampen vibration. However, only minimal damping can be added if the damper attachment point is close to the end of the cable. For long cables, passive dampers may provide insufficient supplemental damping to eliminate vibration problems. A recent study by the authors demonstrated that "smart" semi active damping can provide significantly superior supplemental damping for a cable modeled as a taut string. This paper extends the previous work by adding sag, inclination, and axial flexibility to the cable model. The equations of motion are given. A new control-oriented model is developed for cables with sag. Passive, active, and smart (semi active) dampers are incorporated into the model. Cable response is seen to be dramatically reduced by semi active dampers for a wide range of cable sag and damper location.

Raid Karoumi This paper presents a method for modeling and analysis of cable stayed bridges under the action of moving vehicles. Accurate and efficient finite elements are used for modeling the bridge structure. A beam element is adopted for modeling the girder and the pylons. Whereas, a two-node catenary cable element derived using exact analytical expressions for the elastic catenary, is adopted for modeling the cables. The vehicle model used in this study is a so called suspension model that includes both primary and secondary vehicle suspension systems. Bridge damping, bridge-vehicle interaction and all sources of geometric nonlinearity are considered. An iterative scheme is utilized to include the dynamic interaction between the bridge and the moving vehicles. The dynamic response is evaluated using the mode superposition technique and utilizing the deformed dead load tangent stiffness matrix. To illustrate the efficiency of the solution methodology and to highlight the dynamic effects, a numerical example of a simple cable-stayed bridge model is presented.

By Yang-Cheng Wang,[1] Associate Member, ASCE Cables instead of interval piers support cable-stayed bridges, and the bridge deck is subjected to strong axial forces due to the horizontal components of cable reactions. The structural behavior of a bridge deck becomes nonlinear because of the axial forces, large deflection, and nonlinear behavior of the cables and the large deformation of the pylons as well as their interactions. The locations and amplitude of axial forces acting on the bridge deck may depend on the number of cables. A grawal indicated that the maximum cable tension decreases rapidly with the increase in the number of cables. This paper investigates the stability analysis of cable-stayed bridges and considers cable-stayed bridges with geometry similar to those proposed in Agrawal'spaper. A digital computer and numerical analysis are used to examine 2D finite element models of these bridges. The Eigen buckling analysis has been applied to find the minimum critical loads of the cable-stayed bridges. The numerical results indicate that the total cumulative axial forces acting on the bridge girder increase as the number of cables increases, yet because the bridge deck is subjected to strong axial forces, the critical load of the bridges decreases. Increasing the number of cables may not increase the critical load on buckling analysis of this type of bridge. The fundamental critical loads increase if the ratio of *Ip /Ib* increases until the ratio reaches the optimum ratio. If the ratio of *Ip /Ib* is greater than the optimum ratio, depending on the geometry of an individual bridge, the fundamental critical load decreases for all the types of bridges considered in this paper. In order to make the results useful, they have been normalized and represented in graphical form.

D. Janjic; M. Pircher; and H. Pircher During the structural analysis of cable-stayed bridges, some specific problems arise that are not common in other types of bridges. One of these problems is the derivation of an optimal sequence for the tensioning of the stay cables. This paper describes a novel solution to this problem, the unit force method. The method takes into account all relevant effects for the design of cable-stayed bridges, including construction sequence, second-order theory, large displacements, cable sag and time-dependent effects, such as creep and shrinkage or relaxation of prestressing tendons. Information about the implementation of this method into a computer program is given, and an example of a practical application of this method concludes this paper. The method is not restricted to the design of cable-stayed bridges and may well be used for other structural applications in the future.

P SREENIVAS SARMA M S PAVAN KUMAR The paper presents an overview of the performance of steel cable stayed bridges under seismic environment and reports the observation made from the response studies conducted on these bridges. Results obtained from the comparative study carried out on a numerical problem for concrete and steel decks, of a cable stayed steel bridge in a severe zone of India, are also presented.

K SARAVANAN AND SRINIVASAN A number of corrosion protection methods like covering with polyester film, initial coating with acrylic resin, second coating with acrylic resin, third coating with acrylic resin and final coating with a mixture of acrylic resin and sand to give the surface a rough texture. Different methods to protect different cable systems have been discussed. Various methods in different countries is also discussed.

YVES BOURNAND Cable vibration on cable stayed bridges is known since several years, and can be considered today as one of the most critical problems for this type of bridge. Engineers developed some damping devices that has been reviewed here, with a particular point on their installation, fatigue and maintenance. Dampers are submitted to small movements and small loads but with a high number of cycles. Fatigue maintenance is perhaps the two important criteria to be considered.

K V RAMANA REDDY, R RAMESH REDDY AND MANOHAR RAO An attempt is been made to determine the vibrational behavior of the cable stayed bridge under wind loads. The vibrational characteristics such as natural frequencies and damping have been measured and the wind induced vibrational behavior of the structure has been observed. The wind tunnel experiments are carried out on a modal to establish the flutter stability of the bridge and to determine resonance vibrations due to vortex shedding. A comparison is made between calculated and experimental values of natural frequencies and damping.

S V RAJEEVA AND J MOUNESWARA The long span contemporary cable stayed bridges having the range of 150 m to 1500 m are very appealing aesthetically and are also very important life-line structures. It is necessary to study the structural behavior of cable stayed bridge and also the parameters which influence the structural behavior of cable stayed bridges. The parameter which is considered in the study is the moment of inertia of the deck. The influence of this parameter on the design variables like deflection of the deck, moment on the deck and pylon is reported.

K V RAMANA REDDY, R RAMESH REDDY AND J NANAK RAM This paper presents the results of an aeroelastic investigation on the dynamic response of a long span cable stayed bridge due to wind loading. The present investigation used a box girder section and dealt concerned with the effects of mass, yawed winds and erection phases on the response of the bridge to buffeting and torsional instability. The model is built to a geometrical scale of 1:50 and is tested in a low speed wind tunnel. The measurements are made of vortex shedding response, buffeting response and torsional instability response as a function of wind speed and compared with the calculated values. Another phase of this experiment investigated the response of the model to a 30 degree yawed wind.

K V RAMANA REDDY, R RAMESH REDDY AND M BHASKAR The fatigue behavior of "Ganga Bridge" is been investigated. Fatigue tests are conducted on the bridge deck and on the cable stays of the bridge. Based on the design and analysis of this cable stayed bridge, fatigue rules were formulated. The test results are compared with the fatigue rules and concluded that it is safe against fatigue for the first 50 years of life.

Chapter 4
Analysis of Cable Stayed Bridge Using SAP-2000

4.1 INTRODUCTION

SAP 2000 is a finite element based structural analysis program with special purpose features for structural design and analysis of building systems. Everything you need is integrated into one versatile analysis and design system with one user interface. Creation and modification of the model, execution of the analysis, and checking and optimization of the design can be done through this single interface. Graphical displays of the results, including real-time display of time-history displacements, are easily produced. There are no external modules to maintain and no worries about data transfer between modules. The effects on one part of the structure from changes in another part are instantaneous and automatic. SAP2000 feature sophisticated capabilities, such as fast equation solvers, force and displacement loading, non-prismatic frame elements, highly accurate shell elements, Eigen and Ritz dynamic analysis, multiple coordinate systems for skewed geometry, many different constraint options, the ability to merge independently defined meshes, a fully-coupled 6-by-6 spring stiffness, and the option to combine or envelope multiple dynamic analyses in the same run. Compare to all other finite element soft wares SAP 2000 has a very powerful and user-friendly pre and post processor interface.

The program is structured to support a wide variety of the latest national and international design codes for the automated design and check of concrete and steel frame members. The presentation of the output is clear and concise. The information is in a form that allows the engineer to take appropriate remedial measures in the event of member overstress. Backup design information produced by the program is also provided for convenient verification of the results. English as well as SI and MKS metric units can be used to define the model geometry and to specify different parameters.

4.2 OBJECTS AND ELEMENTS

The physical structural members in a SAP2000 model are represented by objects. Using the graphical user interface, you “draw” the geometry of an object, then “assign” properties and loads to the object to completely define the model of the physical member.

The following object types are available, listed in order of geometrical dimension:

4.2.1 Point objects:

Joint objects: These are automatically created at the corners or ends of all other types of objects below, and they can be explicitly added to model supports or other localized behavior.
Grounded (one-joint) link objects: Used to model special support behavior such as isolators, dampers, gaps, multilinear springs, and more. These are not covered in this manual
Frame/cable objects: Used to model beams, columns, braces, trusses, and/or cable members
Connecting (two-joint) link objects: Used to model special member Behavior such as isolators, dampers, gaps, multilinear springs, and more. Un-like frame/cable objects, connecting link objects can have zero length.
Area objects: Used to model walls, floors, and other thin-walled members, as well as two-dimensional solids (plane stress, plane strain, and axisymmetric solids). Only shell-type area objects are covered in this manual
Solid objects: Used to model three-dimensional solids.

4.2.2 Frame Element

The Frame element is used to model beam-column and truss behavior in planar and three-dimensional structures. The frame element can also be used to model cable behavior when nonlinear properties are added (e.g., tension only, large deflections). The Frame element uses a general, three-dimensional, beam-column formulation, which includes the effects of biaxial bending, torsion, axial deformation, and biaxial shear deformations.
Structures that can be modeled with this element include:

- Three-dimensional frames
- Three-dimensional trusses
- Planar frames
- Planar grillages
- Planar trusses
- Cables

A Frame element is modeled as a straight line connecting two points. In the graphical user interface, you can divide curved objects into multiple straight objects, subject to your specification. Each element has its own local coordinate system for defining section properties and loads, and for interpreting output. Each Frame element may be loaded by self-weight, multiple concentrated loads, and multiple distributed loads. The Frame element activates all six degrees of freedom at both of its connected joints. If you want to model truss or cable elements that do not transmit moments at the ends, you may either:

- Set the geometric Section properties **j**, **i33**, and **i22** all to zero (**a** is non-zero; **as2** and **as3** are arbitrary), or
- Release both bending rotations, R2 and R3, at both ends and release the torsional rotation, R1, at either end

Each Frame element has its own element local coordinate system used to define section properties, loads and output. The axes of this local system are denoted 1, 2 and 3. The first axis is directed along the length of the element; the remaining two axes lie in the plane perpendicular to the element with an orientation that you specify.
The material properties for the Section are specified by reference to a previously defined Material. The material properties used by the Section are:

- The modulus of elasticity, **e1**, for axial stiffness and bending stiffness;
- The shear modulus, **g12**, for torsional stiffness and transverse shear stiffness; this is computed from **e1** and the Poisson's ratio, **u12**
- The mass density (per unit of volume), **m**, for computing element mass;
- The weight density (per unit of volume), **w**, for computing Self-Weight Load.
- The design-type indicator, **ides**, that indicates whether elements using this Section should be designed as steel, concrete, or neither (no design).

Six basic geometric properties are used, together with the material properties, to generate the stiffnesses of the Section. These are:

- The cross-sectional area, **a**. The axial stiffness of the Section is given by **a.e1;**
- The moment of inertia, **i33**, about the 3 axis for bending in the 1-2 plane, and the moment of inertia, **i22**, about the 2 axis for bending in the 1-3 plane. The corresponding bending stiffnesses of the Section are given by **i33.e1**and **i22.e1;**
- The torsional constant, **j**. The torsional stiffness of the Section is given by **j.g12**.
- The shear areas, **as2** and **as3**, for transverse shear in the 1-2 and 1-3 planes, respectively.

The corresponding transverse shear stiffnesses of the Section are given by **as2.g12** and **as3.g12**. Setting **a**, **j**, **i33**, or **i22** to zero causes the corresponding section stiffness to be zero. For example, a truss member can be modeled by setting **j** = **i33** = **i22** = 0, and a planar frame member in the 1-2 plane can be modeled by setting **j** = **i22** = 0. Setting **as2** or **as3** to zero causes the corresponding transverse shear deformation to be zero. In effect, a zero shear area is interpreted as being infinite. The transverse shear stiffness is ignored if the corresponding bending stiffness is zero.

A Frame element is modeled as a straight line connecting two points. In the graphical user interface, one can divide curved objects into multiple straight objects, subject to ones specification. Each element has its own local coordinate system for defining section properties and loads, and for interpreting output. Each Frame element may be loaded by self-weight, multiple concentrated loads, and multiple distributed loads. The Frame element activates all six degrees of freedom at both of its connected joints. If one wants to model truss or cable elements that do not transmit moments at the ends, one may either:

Set the geometric Section properties **j**, **i33**, and **i22** all to zero (**a** is non-zero; **as2** and **as3** are arbitrary), or Release both bending rotations, R2 and R3, at both ends and releases the torsional rotation, R1, at either end.

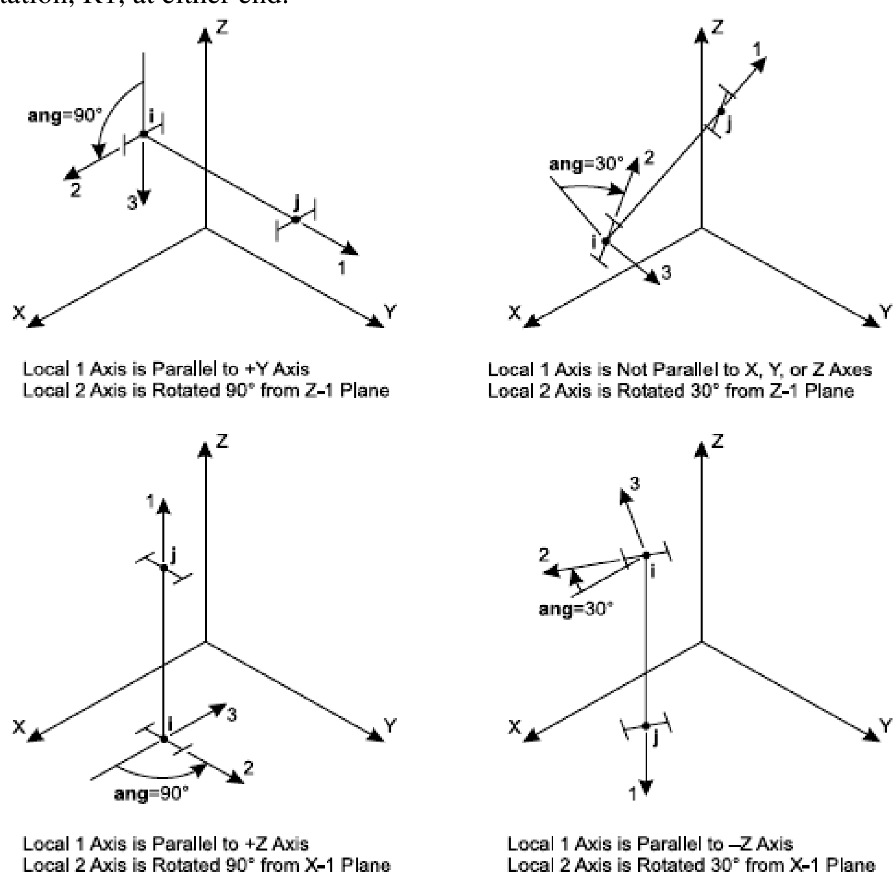


Fig. 4.1: **The Frame Element Coordinate Angle**

Each Frame element has its own element local coordinate system used to define section properties, loads and output. The axes of this local system are denoted 1, 2 and 3. The first axis is directed along the length of the element; the remaining two axes lie in the plane perpendicular to the element with an orientation that one specifies.

4.2.3 Shell Element

In SAP 2000 the shell element is a three or four node formulation that combines separate membrane and plate- bending behavior. The membrane behavior uses an isoparametric formulation that includes transnational in-plane stiffness components and rotational stiffness component in the direction normal to the plane of the element. By default, a thick plate (Mindlin) formulation is used which includes the effects of transverse shearing deformation. Optionally, you may choose a thin-plate (Kirchhoff) formulation that neglects transverse shearing deformation.

Structures that can be modeled with this element include:

- Three-dimensional shells, such as tanks and domes
- Plate structures, such as floor slabs
- Membrane structures, such as shear walls

For each Shell element in the structure, you can choose to model pure membrane, pure plate, or full shell behavior. It is generally recommended that we use the full shell behavior unless the entire structure is planar and is adequately restrained.

Each Shell element has its own local coordinate system for defining Material properties and loads, and for interpreting output. Each element may be loaded by gravity or uniform load in any direction.

A variable, four to eight point numerical integration formulations is used for the shell stiffness. Stresses and internal forces and moments, in the element local coordinate system, are evaluated at the 2 by 2 Gauss integration points and extrapolated to the joints of the element. An approximate error in the element stresses or internal forces can be estimated from the difference in values calculated from different elements attached to a common joint. This will give an indication of the accuracy of a given finite-element approximation and can then be used as the basis for the selection of a new and more accurate finite element mesh.

Each Shell element may have either of the following shapes.

- Quadrilateral, defined by the four joints **j1**, **j2**, **j3**, and **j4**.
- Triangular, defined by the three joints **j1**, **j2**, and **j3**.

The quadrilateral formulation is the more accurate of the two. The triangular element is recommended for transitions only. The stiffness formulation of the three-node element is reasonable; however, its stress recovery is poor.
The locations of the joints should be chosen to meet the following geometric conditions:

- The inside angle at each corner must be less than 180°. Best results for the quadrilateral will be obtained when these angles are near 90°, or at least in the range of 45° to 135°.
- The aspect ratio of an element should not be too large. For the triangle, this is the ratio of the longest side to the shortest side. For the quadrilateral, this is the ratio of the longer distance between the midpoints of opposite sides to the shorter such distance. Best results are obtained for aspect ratios near unity, or at least less than four. The aspect ratio should not exceed ten.
- For the quadrilateral, the four joints need not be coplanar. A small amount of twist in the element is accounted for by the program. The angle between the normal at the corners gives a measure of the degree of twist. The normal at a corner is perpendicular to the two sides that meet at the corner. Best results are obtained if the largest angle between any pair of corners is less than 30°. This angle should not exceed 45°.

The Shell element always activates all six degrees of freedom at each of its connected joints. When the element is used as a pure membrane, you must ensure that restraints or the other supports are provided to the degrees of freedom for normal translation and bending rotations. When the element is used as a pure plate, you must ensure that restraints or other supports are provided to the degrees of freedom for in-plane translations and the rotation about the normal. The use of the full shell behavior (membrane plus plate) is recommended for all three-dimensional structures.
The Shell element stresses are the forces-per-unit-area that act within the volume of the element to resist the loading. These stresses are:

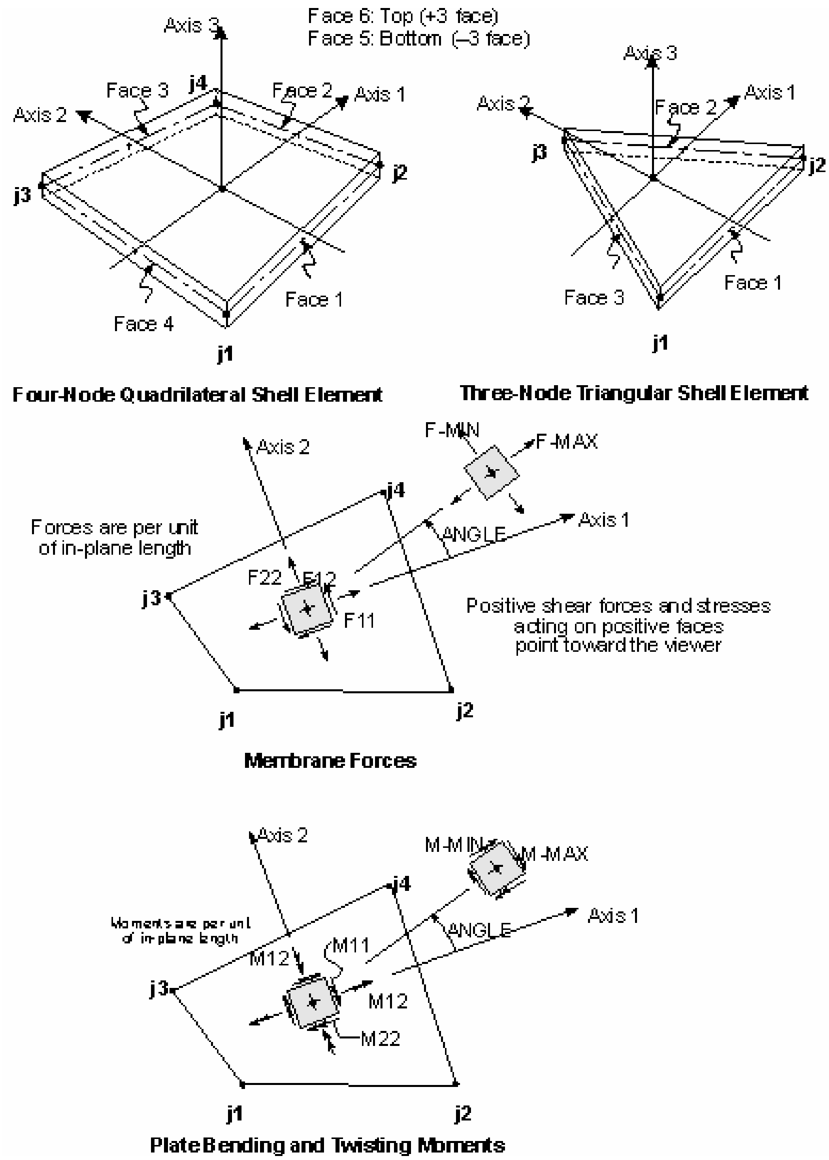


Fig. 4.2 Types of Shell Elements and Internal Forces

- In-plane direct stresses: S11 and S22
- In-plane shear stress: S12
- Transverse shear stresses: S13 and S23
- Transverse direct stress: S33 (always assumed to be zero)

The three in-plane stresses are assumed to be constant or to vary linearly through the element thickness. The two transverse shear stresses are assumed to be constant through the thickness. The actual shear stress distribution is parabolic, being zero at the top and bottom surfaces and taking a maximum or minimum value at the midsurface of the element.

The Shell element internal forces (also called stress resultants) are the forces and moments that result from integrating the stresses over the element thickness. These internal forces are:

- Membrane direct forces: F11 and F22
- Membrane shear force: F12
- Plate bending moments: M11 and M22
- Plate twisting moment: M12
- Plate transverse shear forces: V13 and V23

It is very important to note that these stress resultants are forces and moments per unit of in-plane length. They are present at every point on the mid-surface of the element.

4.3 BRIDGE ANALYSIS

Bridge analysis can be performed to compute influence lines for traffic lanes on bridge structures and to analyze these structures for the response due to vehicle live loads.

Bridge Analysis can be used to determine the response of bridge structures due to the weight of Vehicle live loads. Considerable power and flexibility is provided for determining the maximum and minimum displacements and forces due to multiple-lane loads on complex structures, such as highway interchanges. The effects of Vehicle live loads can be combined with static and dynamic loads, and envelopes of the response can be computed. The bridge to be analyzed is modeled with Frame elements representing the super-structure, substructure and other components of interest. Displacements, reactions, spring forces, and Frame-element internal forces can be determined due to the influence of Vehicle live loads. Other element types (Shell, Plane, Solid and NLLink) may be used, they contribute to the stiffness of the structure, but they are not analyzed for the effect of Vehicle load. Lanes are defined on the superstructure that represent where the live loads can act. These Lanes need not be parallel nor of the same length, so that complex traffic patterns may be considered. One may select Vehicle live loads from a set of standard highway and railway Vehicles, or one may create one's own Vehicle live loads. Vehicles move in both directions along each Lane of the bridge. Vehicles are automatically located at such positions along the length of the Lanes to produce the maximum and minimum response quantities throughout the structure. Each Vehicle live load may be allowed to act on every lane or be restricted to certain lanes. The program can automatically find the maximum and minimum response quantities throughout the structure due to placement of different Vehicles in different Lanes. For each maximum or minimum extreme response quantity, the corresponding values for the other components of response can also be computed. The procedure to perform a Bridge Analysis is to:

- Model the structural behavior of the bridge with Frame elements
- Define traffic Lanes describing where the Vehicle live loads act
- Define the different Vehicle live loads that may act on the bridge
- Define Vehicle Classes (groups) containing one or more Vehicles that must be considered interchangeably
- Define Moving-Load Analysis Cases that assign Vehicle Classes to act on the traffic Lanes in various combinations
- Specify for which joints and Frame elements the Moving Load response is to be calculated the most extreme (maximum and minimum) displacements, reactions, and spring Forces, and Frame element internal forces are automatically computed for each Moving-Load Analysis Case defined.

4.3.1 Modeling of Bridge Superstructure using Frame Elements

In simple cases define a "two-dimensional" model with longitudinal elements representing the superstructure and roadway, and vertical elements representing the piers and supports. For curved bridge structures these Frame elements need not exist in a single plane. Fig. 3.3 shows the curve bridge model using frame elements. Elements directed in the third, transverse direction might also be used for modeling the bents and other features. The results of the Bridge Analysis will report the Frame element internal forces and moments, which can then be used to design the actual sections. Moving-load response will only be calculated for those elements one specifically request.

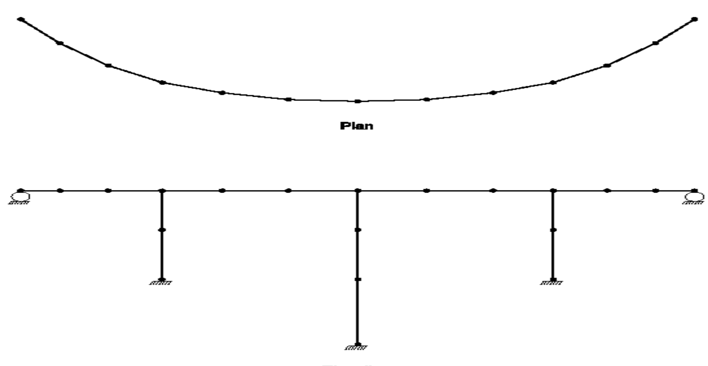


Fig. 4.3 Frame Element as Bridge Superstructure

4.3.2 Supports

Supports can be modeled using either springs or restraints. Moving-load response will only be calculated for those springs or restraints one specifically request.

4.3.3 Bearings and Expansion Joints

Effective modeling of support conditions at bearings and expansion joints requires careful consideration of the continuity of each translational and rotational component of displacement. Continuous components require that the corresponding degrees-of-freedom remain connected across the bearing or expansion joint.

Degrees-of-freedom representing discontinuous components must be disconnected. One can achieve this by two principal methods:

(1) Attaching elements to separate joints at the same location (which automatically disconnects all degrees-of-freedom between the elements) and constraining together the connected degrees-of-freedom using an Equal or Local Constraint, or

(2) Attaching several elements to a common joint (which automatically connects the degrees-of-freedom between the elements) and using Frame element end releases to free the unconnected degrees-of-freedom.

Both methods are acceptable for static analysis. For dynamic analysis, method (1) is recommended since method (2) does not properly distribute the mass on either side of the joint.

Typically the vertical and transverse translations and the torsional rotation would be connected, while the longitudinal translations and the bending and in-plane rotations would be disconnected. However, the appropriate use of constrained or released degrees-of-freedom depends on the details of each individual bearing or joint as shown in Fig.

Shell, Plane, Asolid, Solid, and Link elements should not generally be used in models subjected to Vehicle loads. If one needs to use these types of elements, one should do so with caution and with complete understanding of the following implications:

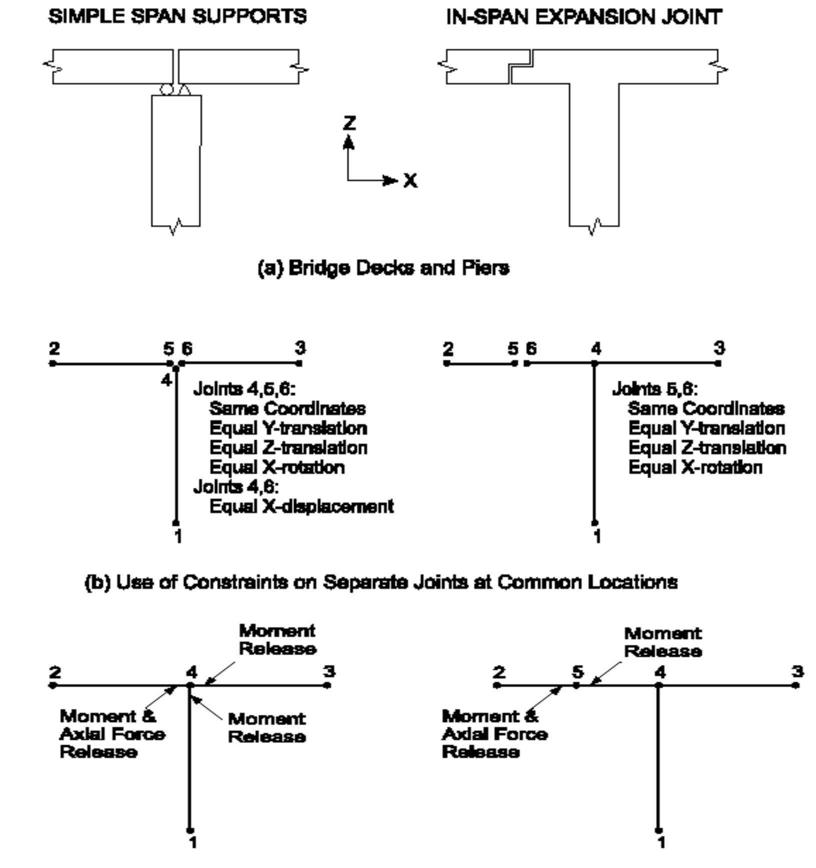


Fig. 4.4 Modeling of Bearing and Expansion joints

Vehicle live loads can only be applied to Frame elements. Thus live loads cannot be represented as acting directly on bridge decks modeled with Shell or other element types.

All elements present in the structure contribute to the stiffness and may carry part of the load. However, element internal forces (stresses) due to Vehicle live loads are computed only for Frame elements. Therefore, the presence of other element types may result in an underestimate of the internal forces in Frame elements if these are intended to represent the complete behavior of the sub-structure or superstructure. The corresponding response in the other element types will not be reported. This approach may be unconservative for all element types.

4.3.4 Roadways and Lanes

The Vehicle live loads are considered to act in traffic Lanes transversely spaced across the bridge roadway. These Lanes are supported by Frame elements representing the bridge deck.

The number of Lanes and their transverse spacing can be chosen to satisfy the appropriate design-code requirements.

4.3.4.1 Roadways

Typically each roadway is modeled with a single string (or chain) of Frame elements running along the length of the roadway. These elements should possess Section properties representing the full width and depth of the bridge deck. They are modeled as a normal part of the overall structure and are not explicitly identified as being roadway elements.

4.3.4.2 Lanes

A traffic Lane on a roadway has its length represented by a consecutive set of some or all of the roadway elements. The transverse position of the Lane center line is specified by its eccentricity relative to the roadway elements. Each Lane across the roadway width will usually refer to the same set of roadway elements, but will typically have a different eccentricity. The eccentricity for a given Lane may also vary along the length.

A Lane is thus defined by listing, in sequence, the labels of a chain of Frame elements that already exist as part of the structure. Each Lane is said to "run" in a particular direction, namely from the first element in the listed sequence to the second element, and so on, to the last element. This direction may be the same or different for different Lanes using the same roadway elements, depending on the order in which each Lane is defined. It is independent of the direction that traffic travels.

4.3.5 Eccentricities

The sign of a Lane eccentricity is defined as follows: in an elevation view of the bridge where the Lane runs from left to right, Lanes located behind the roadway elements have positive eccentricity. Alternatively, to a driver traveling on the roadway in the direction that the Lane runs, a Lane to the left of the roadway elements has a positive eccentricity.

The use of eccentricities is primarily important for the determination of axial torsion in the bridge deck and transverse bending in the substructure; secondary effects may also be found in more complex structures. Although the modeling of lane eccentricities is generally realistic and advantageous, some savings in computation time, memory requirements, and disk storage space can be realized by using zero eccentricities for all elements in all Lanes.

4.4 MODELING OF CABLE-STAYED BRIDGE

4.4.1 Geometry of Bridge

Central Span of the Bridge	160 Mts
Width of Bridge	13.5 Mts (2-Lanes)
Side Span	64 mts
Height of Pylon	32 mts
No. of Cables	30 nos.
Cable Arrangement	Semi Harp Type

The following consists of the steps carried out to model the cable-stayed bridge using SAP2000.

To start with SAP2000 modeling; the default SAP-2000 GUI (Fig. 4.5) is displayed on the screen.

From the file menu click on the new model option. New model dialogue box appears on the screen, to start with the modeling click on the "grid only" option so that there is flexibility of modeling. Set the unit to kN, m, C.

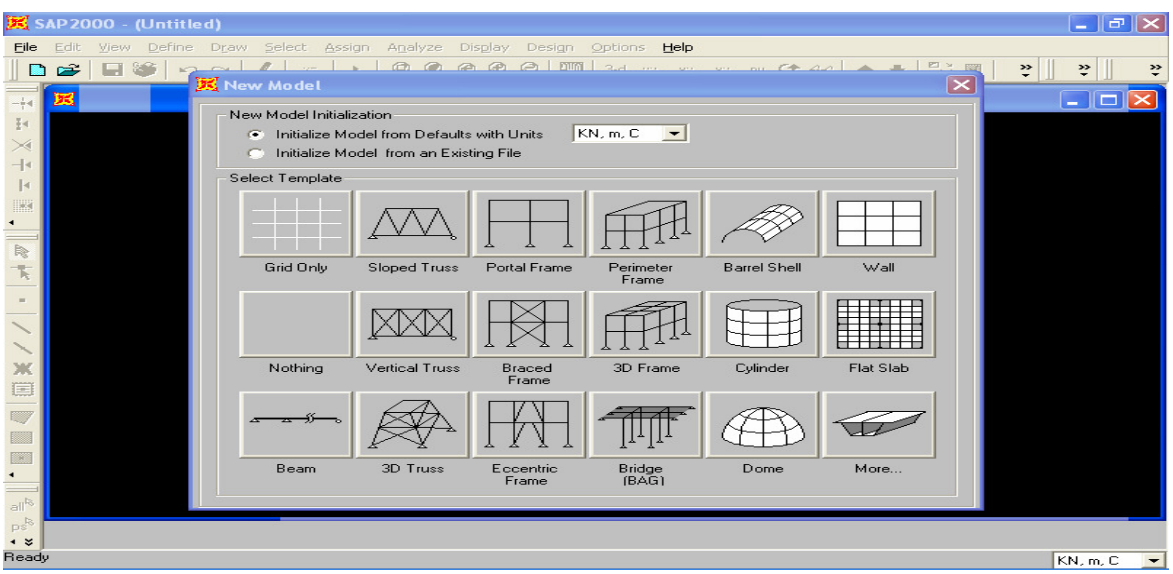


Fig 4.5 Starting With SAP-2000

On clicking "grid only", the grid system dialogue box appears (Fig 4.6); keep the default values to start with so that they can be edited later to the requirement. The grid lines can be edited using edit grid line dialogue box (Fig.4.7).

Enter the values of coordinates in X, Y and Z axis

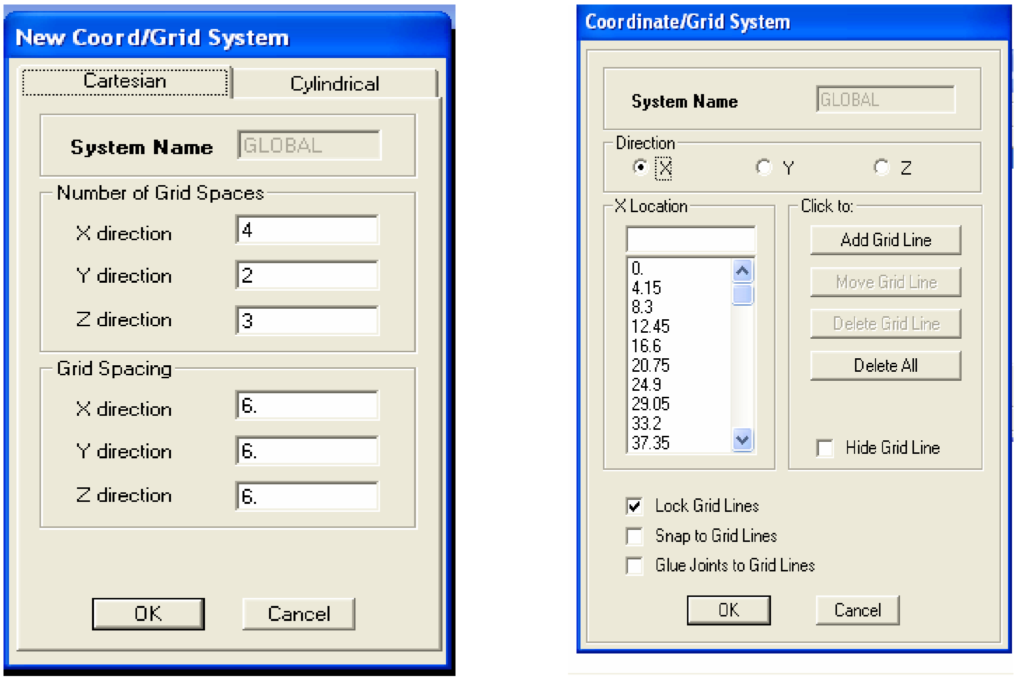


Fig. 4.6: **Grid System Dialogue Box**

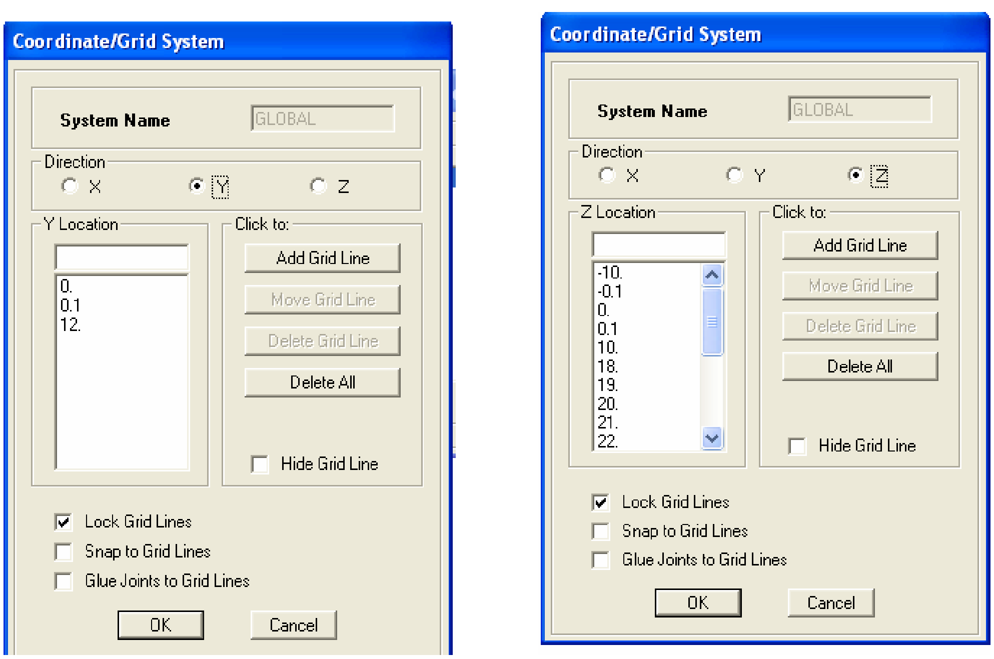


Fig. 4.7: **Edit Grid Dialogue Box**

The Cables are attached to the deck at 4.15 m c/c in X direction, the width of the bridge deck is 13.5 m in Y direction, cables are attached to the pylon at 1 m c/c from 18 m to 32 m which is shown in Z direction. The coordinate system can be seen as shown in fig.4.8.

Following gird line appears with the input of above mentioned coordinates.

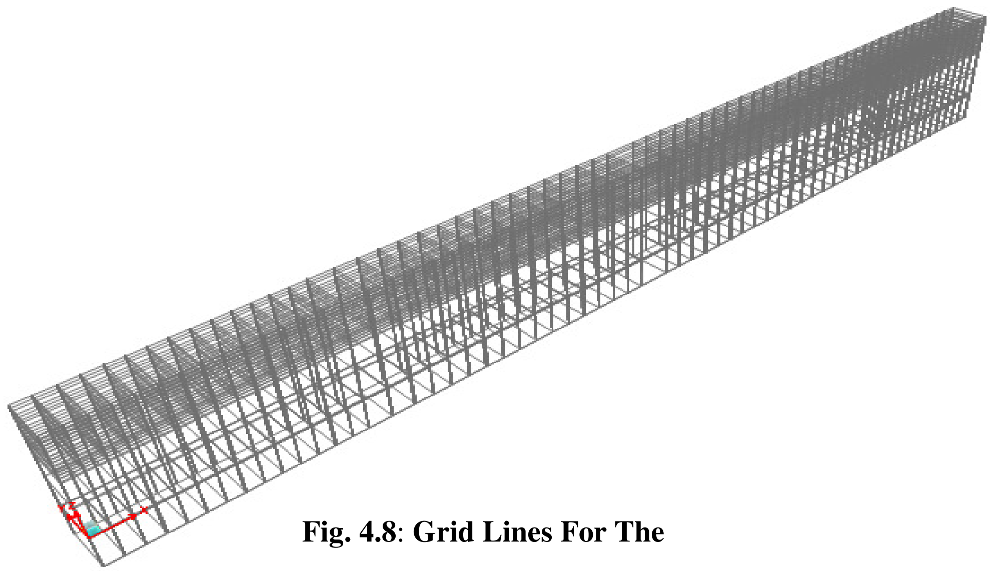


Fig. 4.8: **Grid Lines For The**

After assigning the coordinates the next step to modeling is to define the materials used by the members in the structure.

SAP has default materials line Aluminum, Steel and Concrete in its data base. New materials can be defined if required.

In this particular case two material; Cable and Bearing are required for the modeling of cables and bearing. This is explained in the following:

Enter the Define menu and click on the material to display Define Material Dialogue Box. Click the Add new Material tab to display the following dialogue box (Fig. 4.9).

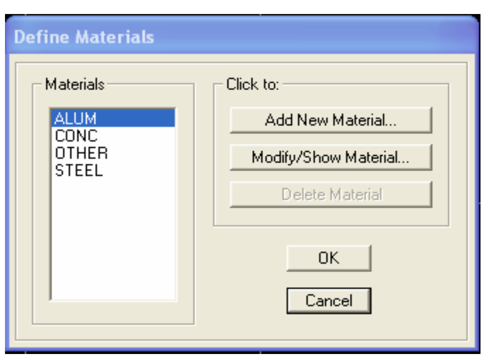


Fig. 4.9: **Define Material Dialogue Box**

Define two new material; Cable and Bearing which will be used in later stage to define cables and bearing (Fig 4.10).

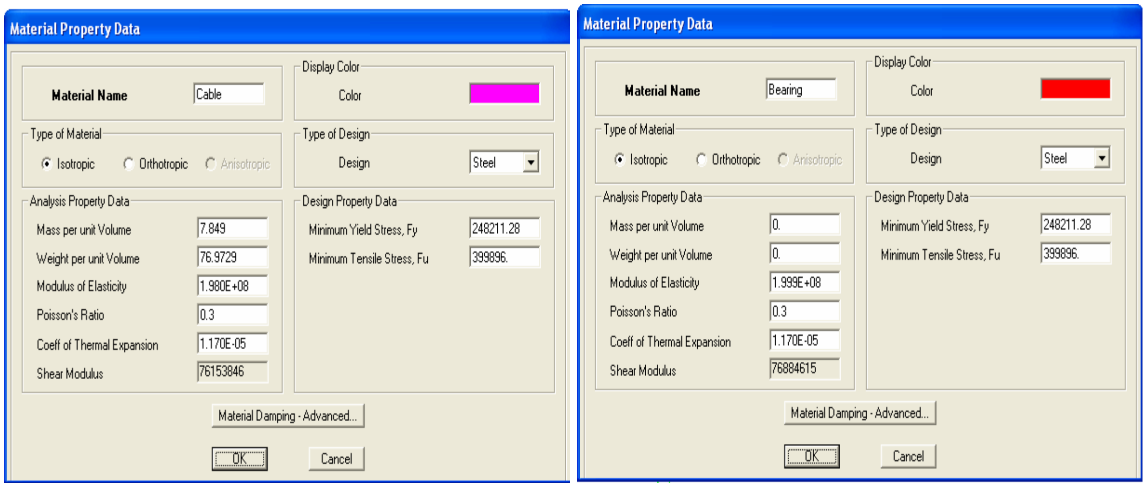


Fig. 4.10: **Define New Material Dialogue Box**

Next in the modeling is to define the elements which are components of cable stayed bridge. Following are the Frame elements which are used in modeling:

- Pylon above Deck
- Pylon below Deck
- Beam Connecting Pylons
- Plate Girder
- Cables

To start with the defining of frame elements, enter the define menu and click frame/cable sections. This will show frame property dialogue box which shown default SAP-2000 section data base.

To add Pylon section click on add New property tab which will display Rectangular section dialogue box;

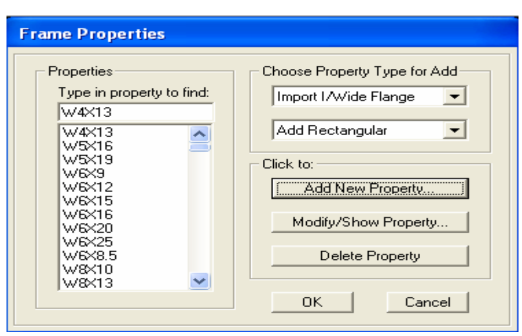


Fig. 4.11: **Database of Inbuilt Frame Sections**

Similarly define Cable Section, Plate Girder Section and Member for Bearing (Fig 4.12).

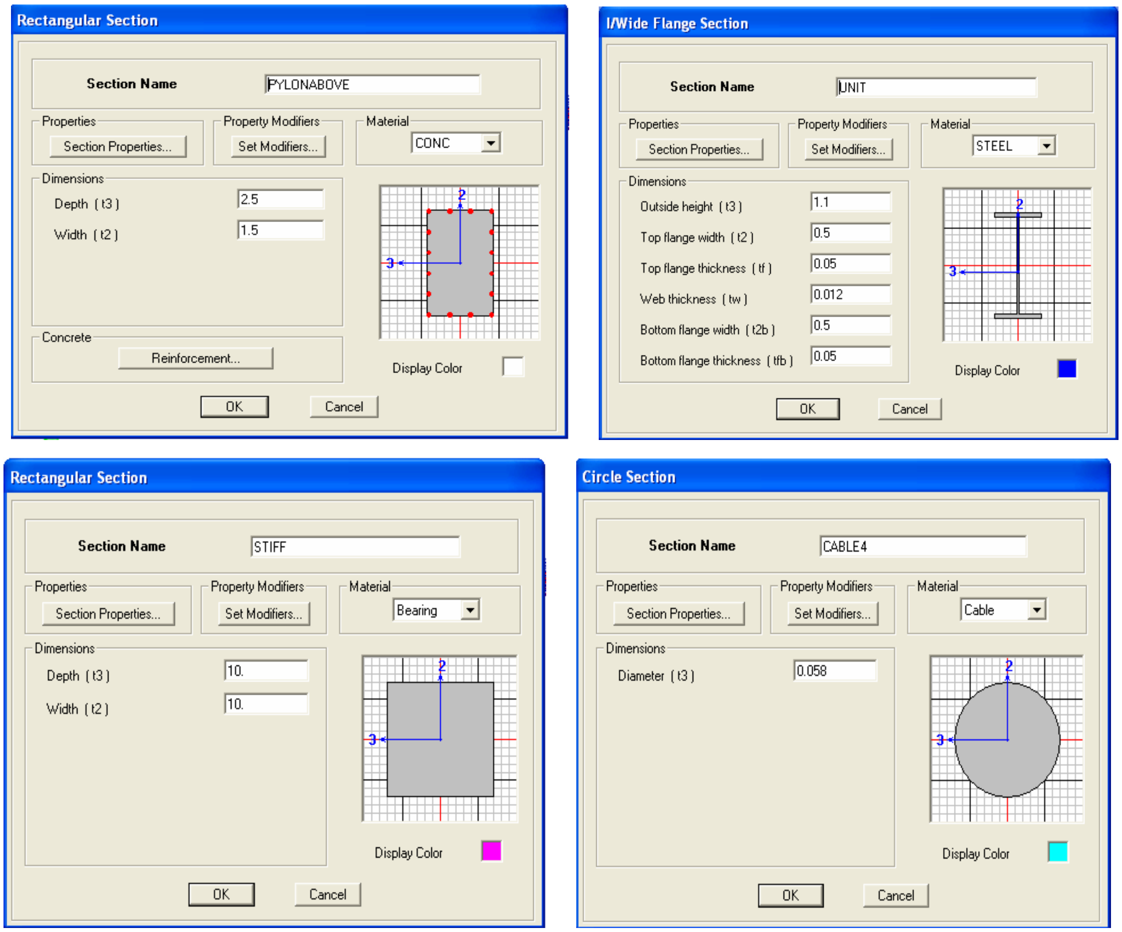


Fig. 4.12: **New Sections**

This completes the pre-requisite for the generating the structure of cable-stayed bridge.

The next step involves assigning the properties to the grid lines which ultimately leads to the required geometry of structure for analysis.

Next is discussed the crucial points in modeling, load Cases, load combinations and analysis cases used in the analysis of bridge.

After assign of properties is complete the structure looks as shown in the following fig

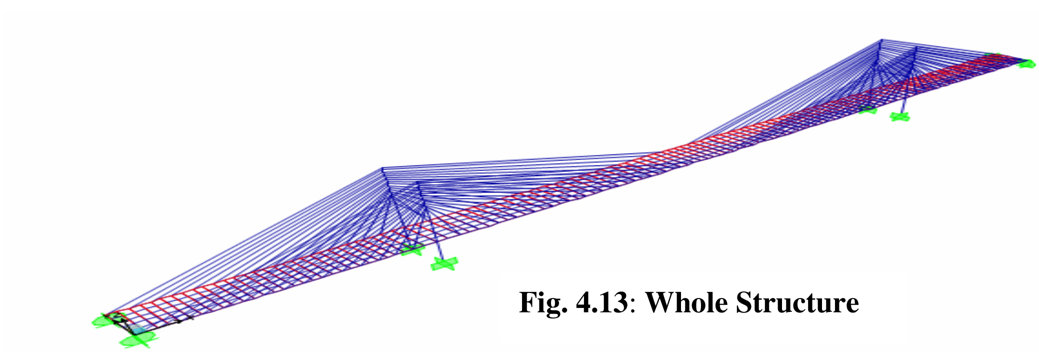


Fig. 4.13: **Whole Structure**

4.4.2 Restraints and Constraints

The ends of the deck are roller supported on all the four sides while the base of pylon is fixed supported.

To assign the required restraints select the joint to be assigned restraints and enter the assign menu, click on joints which leads to joint restraints dialogue box. The first fig (Fig 4.14) shows hinged supports while the second fig shows fixed support.

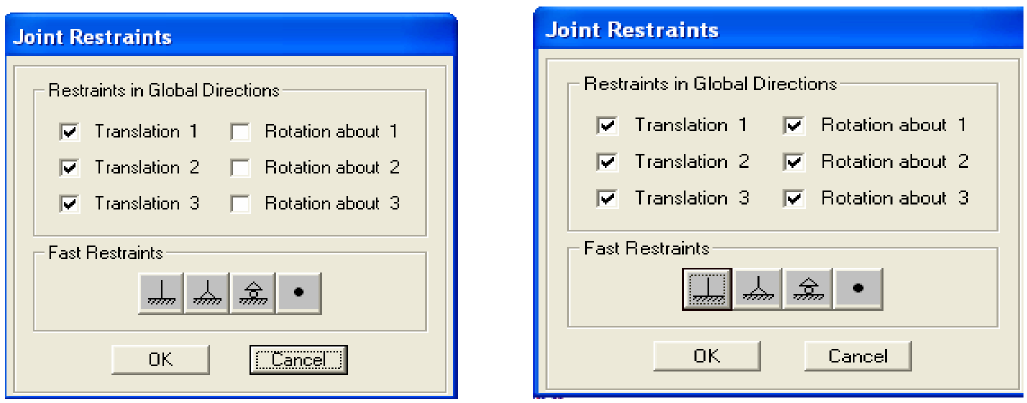


Fig. 4.14: Joint Restraints Dialogue Box

For continuity of each translational and rotational component of displacement at the expansion joint, continuous components are required such that corresponding degree of freedom remains connected across the bearing or expansion joint. This is achieved by attaching elements to separate joints at the same location (which automatically disconnects all degrees-of-freedom between the elements) and constraining together the connected degrees-of-freedom using an Equal or Local Constraint. This is incorporated in this model as shown below:

Enter the define menu, click joint constraints to display "define constraints" dialogue box (Fig 4.15). Choose the type of constraints to define in this case its "Equal" (Fig 4.16). Add a new constraint "Equal1" (Fig 4.17) having translation in X, Y and Z direction checked. Similarly define other 3 constraints for other 3 points.

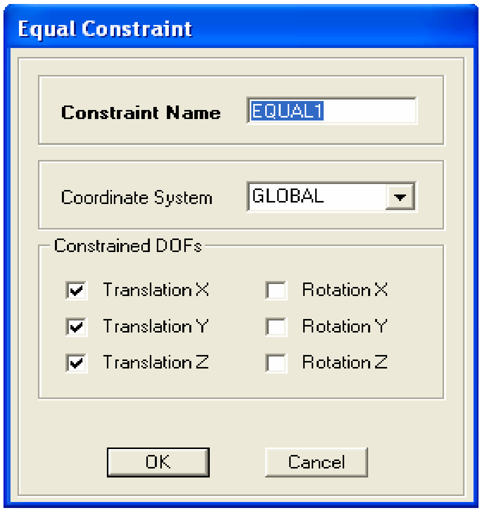


Fig 4.15: Equal Constraints Dialogue Box

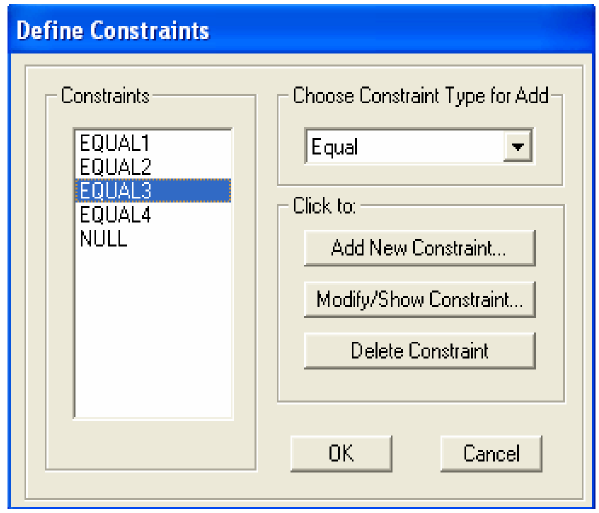


Fig 4.16: **Define Constraints Dialogue Box**

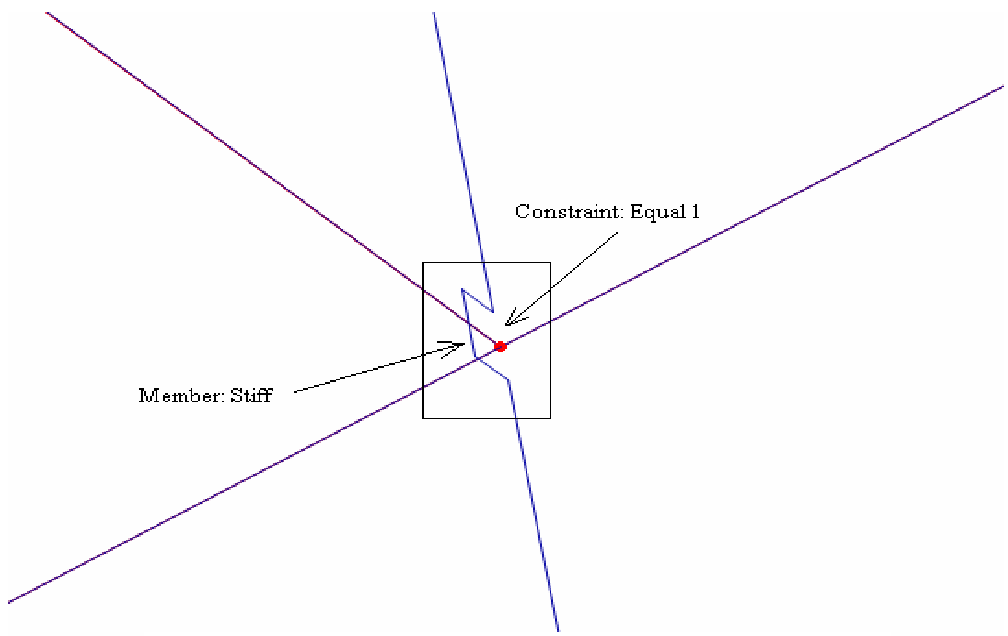


Fig. 4.17: **Equal Constraint Applied to the Structure**

Similarly constraints are provided at all the points where pylon and deck intersects. This arrangement ensures the continuity between the pylon above deck level and the pylon below deck level.

4.4.3 Loads and Load Combinations

The next step to analysis after the generation of model is complete and a property assigned is to define loads and load combinations. Any number of load cases required for the analysis can be defined which later are combined with appropriate scale factors to generate load combinations.

To define a load, enter the "define menu" to click on load cases option. This displays "Define Loads" dialogue box (Fig 4.18). Required no. of load cases can be defined in this menu. All self weight multiplier are set to 0 except for the dead load case.

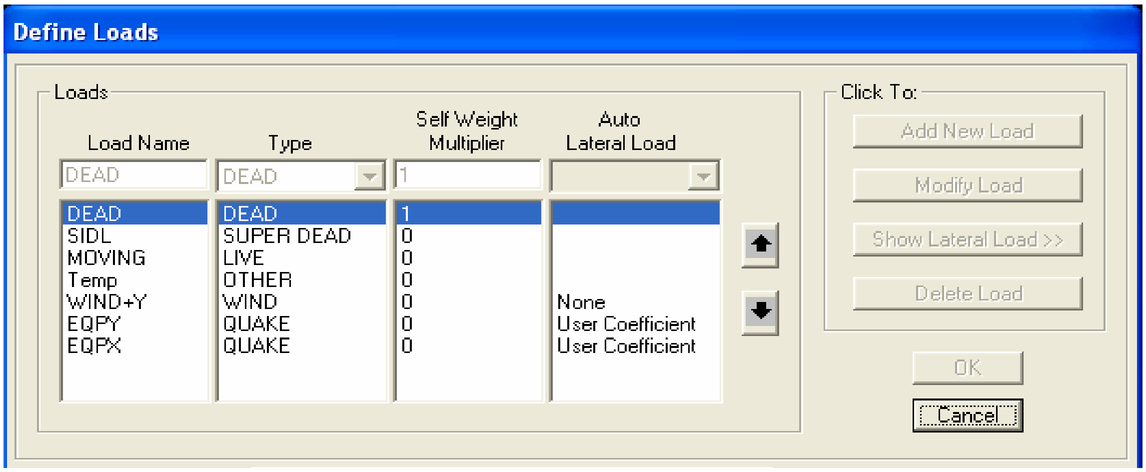


Fig. 4.18: **Define Loads Dialogue Box**

After the defining of primary loads is complete, next step is to move on to defining of load combinations. Required number of load combinations can be defined with different scale factors.

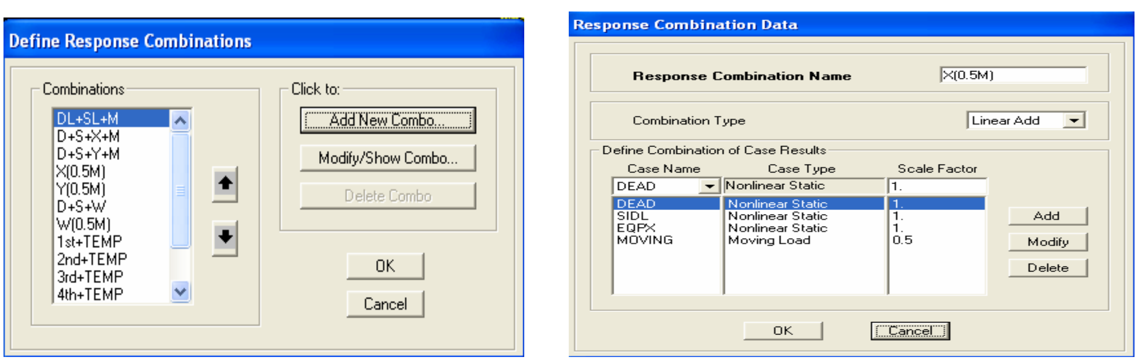


Fig. 4.19: **Load Combinations**

4.4.4 Defining of Bridge Loads

Bridge analysis required defining of bridge loads which comprises the defining of lanes on which vehicles move, the vehicles and the vehicle class. A separate bridge load defining facility is available in SAP-2000.

“Define Menu” has a bridge load menu for the defining of bridge loads. The first step is to define the lanes on which vehicles move. Standard or General vehicles can be defined followed by the vehicle class.

The first step is to define Lanes for the bridge (Fig 4.20). In the present case the bridge is two lane bridge, so two lanes are to be defined. Frame elements are selected and the lanes are assigned to the frame elements with the eccentricities so that the vehicle runs on the specified location on the bridge deck.

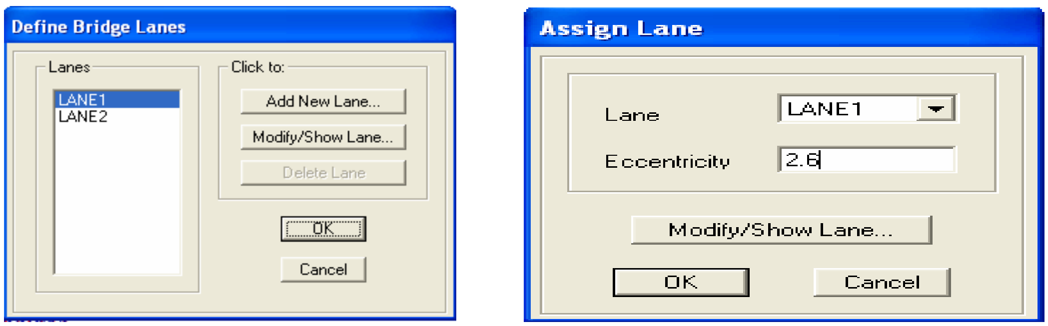


Fig. 4.20: **Define and Assign Bridge Lanes**

Defining a vehicle is the next step to defining bridge load. SAP-2000 gives various standard vehicles compiling to AASHTO specifications.

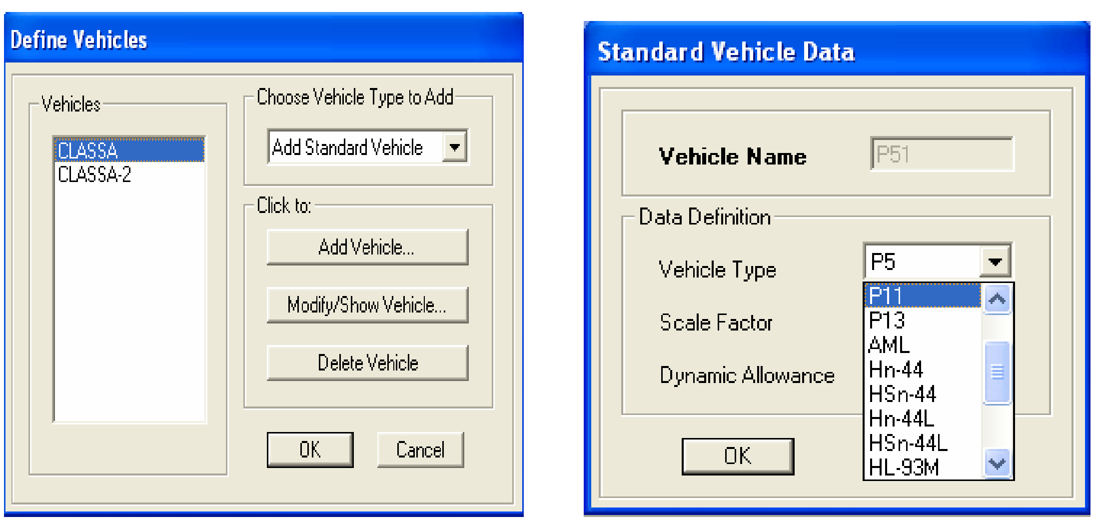


Fig. 4.21: **Define Vehicle**

However SAP-2000 do not support the vehicles specified by Indian Road Congress, so this requires defining a “General Vehicle” instead of “Standard vehicle” (Fig 4.21). In the present study 2 Lanes of class A vehicles are proposed to run along the bridged deck, therefore 2 class A vehicles are defined (Fig 4.22).

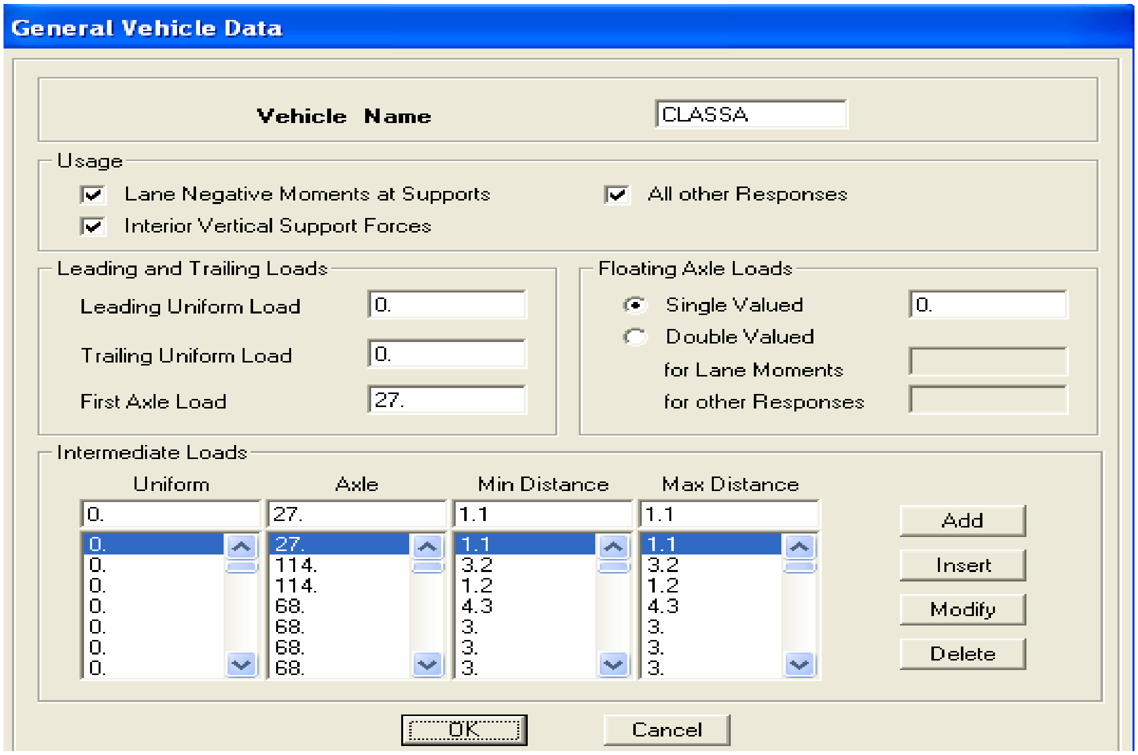


Fig. 4.22: ClassA Vehicle

4.4.5 Analysis Cases

The load cases entered for the analysis are automatically entered as analysis cases which can be defined as Static Linear, Static Non-Linear, Buckling, Response Spectrum, Modal, Time History or Moving (Fig 4.23).

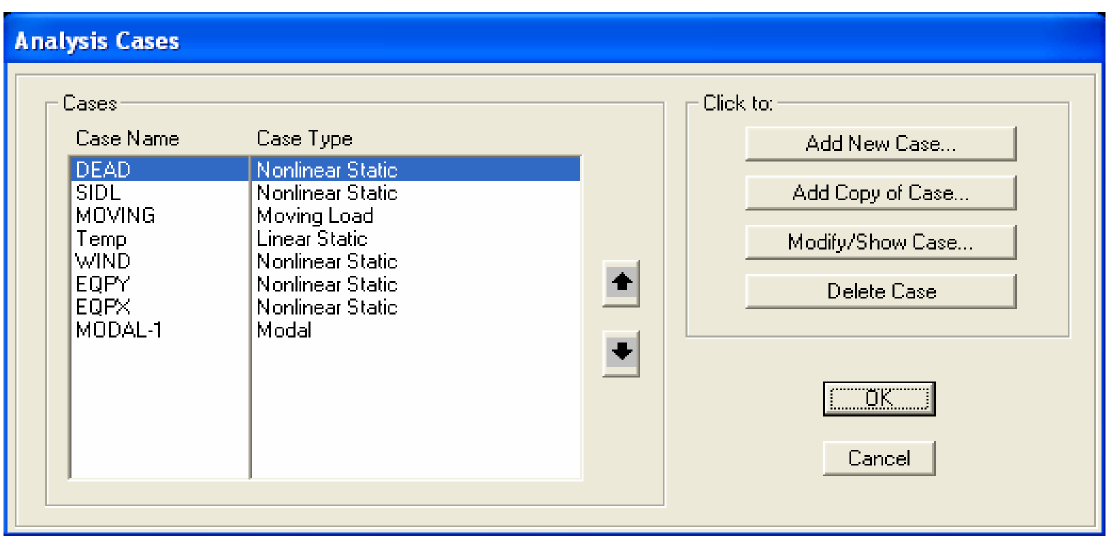


Fig. 4.23: Analysis Case Dialogue Box

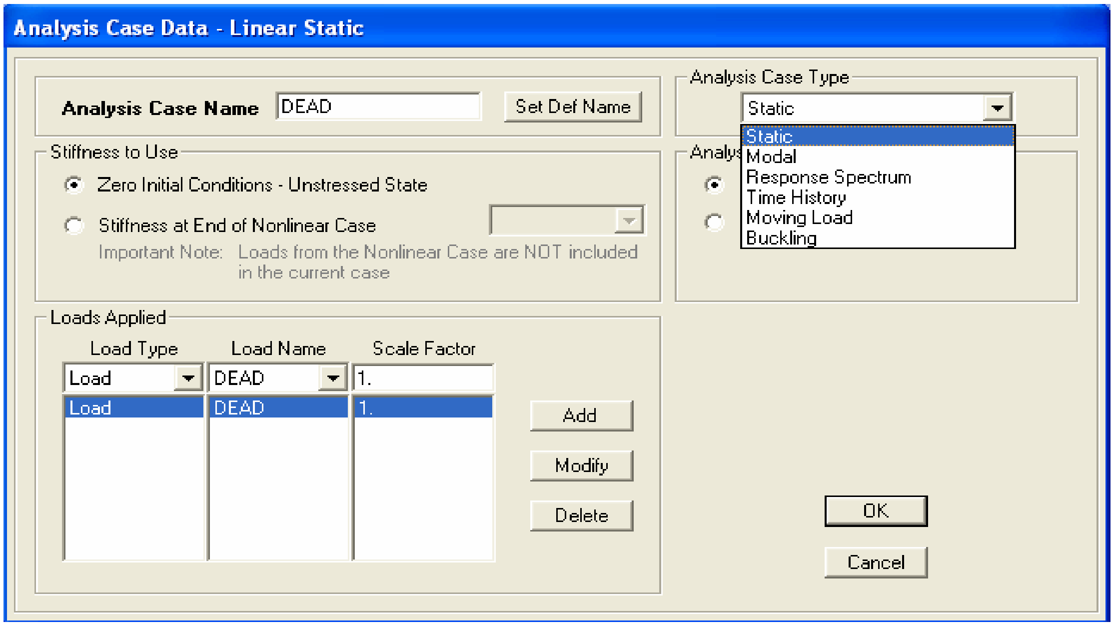


Fig. 4.24: **Linear Static Case**

If the analysis case is non-linear it shows more options than shown above (Fig 4.25). The non-linear analysis case has other parameters like Load Application, Results Saved, Staged Construction and Nonlinear parameters. In the present study the first three parameters are kept default, the only parameter change is the nonlinear parameter.

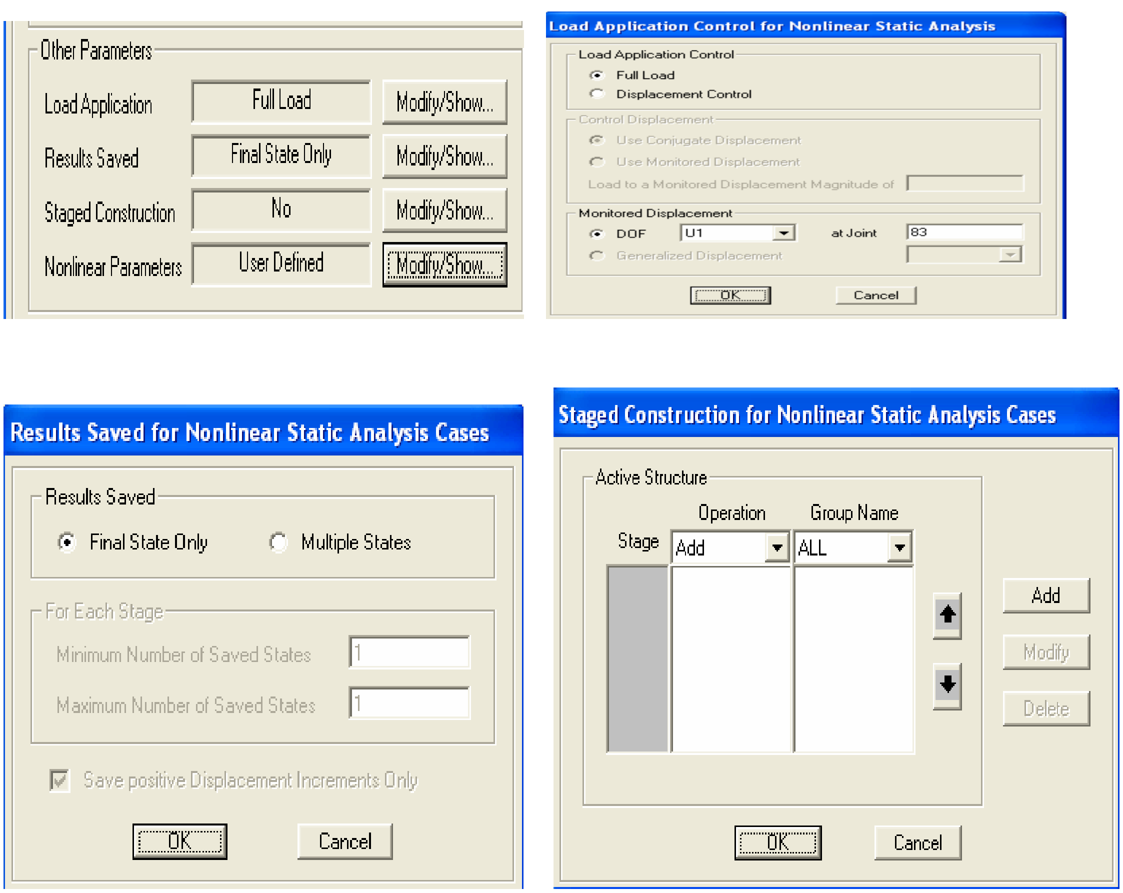


Fig 4.25: Nonlinear Static Analysis

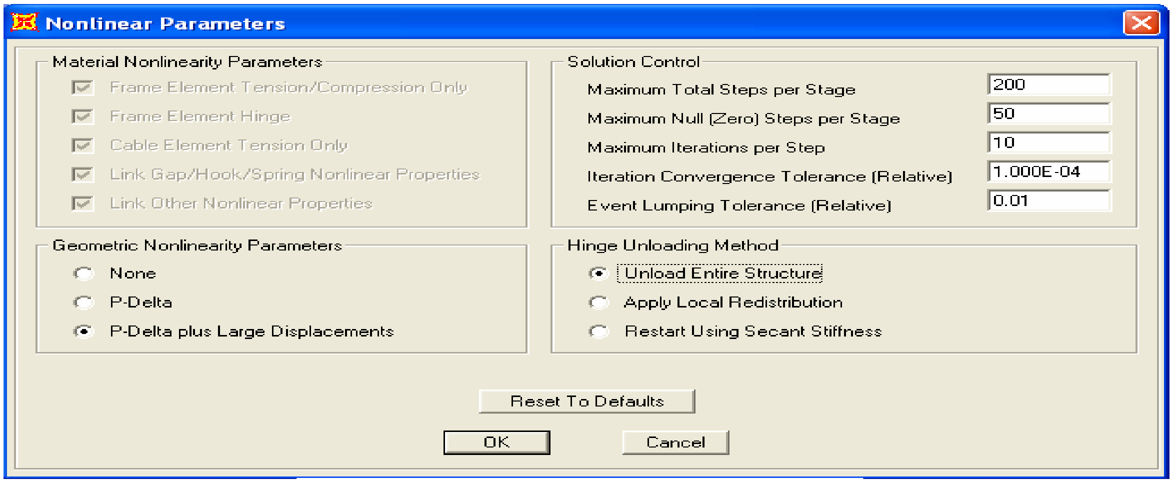


Fig. 4.26: **Nonlinear parameters Options**

This completes the defining of all analysis cases to be used and the modeling of the structure is complete. The structure is ready for analysis.

Analysis is set to run by pressing F5 key. This displays the dialogue box as shown below (Fig 4.26).

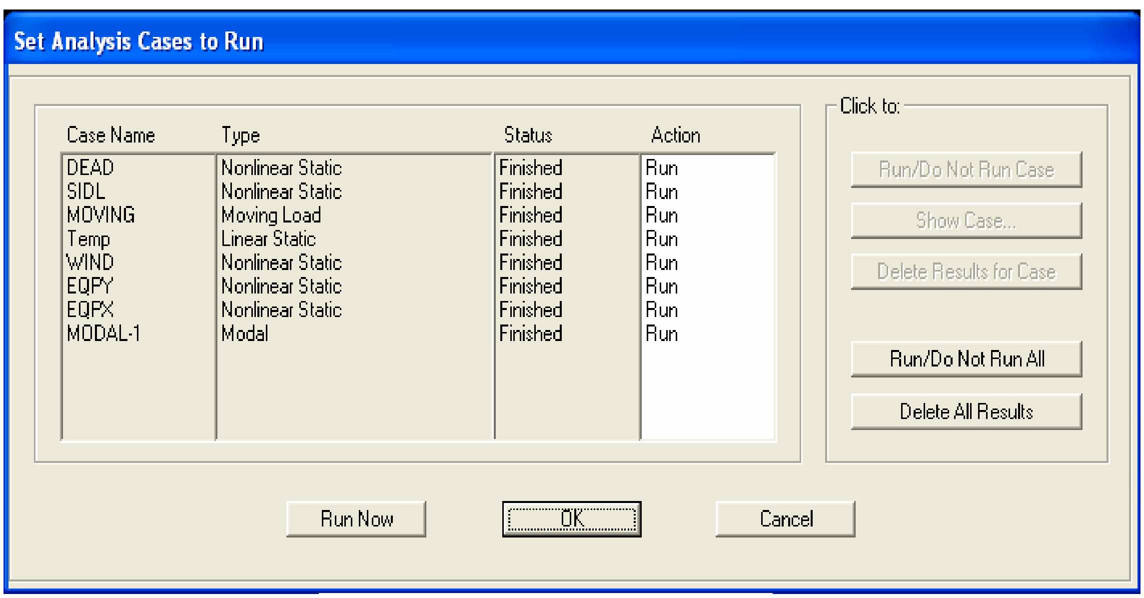


Fig. 4.27: **Run Analysis**

CHAPTER 5

ANALYSIS RESULT AND DESIGN OF COMPONENT

5.1 LOADS

Following loads are considered for the analysis.

i. Dead Load (DL)
ii. Superimposed Dead Load (SIDL)
iii. Vehicle Live Load with impact (Moving Load)
iv. Earthquake Loads
v. Wind Load
vi. Temperature load

5.1.1 Dead Load

Dead Loads of all the components of the bridge like cables, plate girder, pylon etc is considered in this load.

5.1.2 Superimposed Dead Load

The loads like load from Pavement, crash barrier, footpath and railing are considered in this load case.

5.1.3 Vehicle Live Load

The Class-A train and Class- B train of vehicle according to IRC-6 is considered in this load case.

5.1.4 Earthquake Load

The bridge location (Ahemdabad) is in Zone III.

For Zone III, Horizontal seismic co-efficient	= 0.03
Co-efficient for soil foundation system	= 1.0
Importance factor	= 1.5

5.1.5 Wind Load

Wind load on the structure is considered as per IRC-6.

5.2 LOAD COMBINATIONS

DL + SIDL + Moving Load

DL + SIDL + 0.5 Moving Load + Earthquake Load on Longitudinal axis

DL + SIDL + 0.5 Moving Load + Earthquake Load on Transverse axis

DL + SIDL + Moving Load + Wind Load on Longitudinal axis

DL + SIDL + Moving Load + Wind Load on Transverse axis

DL + SIDL + 0.5 Moving Load + Wind Load on Transverse axis

DL + SIDL + Moving Load + Temperature on Stays

5.2.1 Assumptions and Approximation

→ All cables are fixed to the pylon and to the bridge girder at their points of attachment.

→ The cables are assumed to be perfectly flexible. i.e., the flexural stiffness of the cables can be neglected. Flexural rigidity of cables is very small as compared to that of girder and tower elements and hence neglected.

→ Effect of creep in steel is neglected.

→ 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. Podolny has shown that the effect of catenary action for moderate sag to span ratio is not large.

→ The effect of change in geometry, beam-column interaction of girder and pylon elements and the effect of wraping are neglected.

5.3 RESULTS FOR CABLE FORCES AND DESIGN OF CABLES

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

Initial Dia Provided	Area (mm2)	SAP Analysis Load (KN)	Stress (N/mm2)	Check (Stress < 0.42 UTS)
57	2551.76	2791	1094	**Not Ok**
70	3848.45	3614	939	**Not Ok**
63	3117.25	2503	803	**Not Ok**
58	2642.08	1831	693	**Ok**
58	2642.08	1620	613	**Ok**
51	2042.82	1146	561	**Ok**
47	1734.94	933	538	**Ok**
45	1590.43	864	543	**Ok**
41	1320.25	739	560	**Ok**
40	1256.64	732	583	**Ok**
38	1134.11	681	600	**Ok**
36	1017.88	617	606	**Ok**
38	1134.11	677	597	**Ok**
38	1134.11	657	579	**Ok**
58	2642.08	1455	551	**Ok**
58	2642.08	1255	475	**Ok**
56	2463.01	1183	480	**Ok**
42	1385.44	735	531	**Ok**
44	1520.53	867	570	**Ok**
46	1661.90	995	599	**Ok**
48	1809.56	1115	616	**Ok**
50	1963.50	1226	624	**Ok**
52	2123.72	1333	628	**Ok**
54	2290.22	1443	630	**Ok**
54	2290.22	1450	633	**Ok**
54	2290.22	1465	640	**Ok**
56	2463.01	1596	648	**Ok**
56	2463.01	1619	657	**Ok**
54	2290.22	1530	668	**Ok**
75	4417.86	3001	679	**Ok**

Revised Dia (mm)	Revised Area (mm2)	Stress	Check (Stress< 0.42 UTS)	No. Of 7 mm Strands	To Provide
77	4657	599	**Ok**	**121**	**122**
80	5027	719	**Ok**	**131**	**132**
75	4418	567	**Ok**	**115**	**116**
58	2642	693	**Ok**	**69**	**70**
58	2642	613	**Ok**	**69**	**70**
51	2043	561	**Ok**	**53**	**54**
47	1735	538	**Ok**	**45**	**46**
45	1590	543	**Ok**	**41**	**42**
41	1320	560	**Ok**	**34**	**36**

40	1257	583	**Ok**	**33**	**34**
38	1134	600	**Ok**	**29**	**30**
36	1018	606	**Ok**	**26**	**28**
36	1018	665	**Ok**	**26**	**28**
38	1134	579	**Ok**	**29**	**30**
58	2642	551	**Ok**	**69**	**70**
58	2642	475	**Ok**	**69**	**70**
56	2463	480	**Ok**	**64**	**66**
42	1385	531	**Ok**	**36**	**38**
44	1521	570	**Ok**	**40**	**40**
46	1662	599	**Ok**	**43**	**44**
48	1810	616	**Ok**	**47**	**48**
50	1963	624	**Ok**	**51**	**52**
52	2124	628	**Ok**	**55**	**56**
54	2290	630	**Ok**	**60**	**60**
54	2290	633	**Ok**	**60**	**60**
54	2290	640	**Ok**	**60**	**60**
56	2463	648	**Ok**	**64**	**66**
56	2463	657	**Ok**	**64**	**66**
54	2290	668	**Ok**	**60**	**60**
75	4418	679	**Ok**	**115**	**116**

5.4 RESULTS FOR PYLON

Pylon of the bridge is divided into 3 different sections;

Pylon below Deck = 9.9 m

Pylon above Deck = 13.4 m

Pylon above Deck = 5.0 m

Pylon at Top = 1.0 m

5.4.1 Pylon below Deck

Pylon is designed as R.C.C. compression member. The designed forces for the pylon are obtained from SAP Output.

Forces	Value	Load Case
Maximum Axial Force	**36589 kN**	(Dead + SIDL + Wind + Temp)
Maximum Moment about Major Axis	**46087 kNm**	(Dead + SIDL + EqX + Moving)
Maximum Moment about Minor Axis	**39523 kNm**	(Dead + SIDL + EqY + Moving)

Table 5.1: Forces for Pylon below Deck

5.4.2 Pylon above Deck (13.4 m)

Design Forces:

Forces	Value	Load Case
Maximum Axial Force	**27134 KN**	(Dead + SIDL + Wind + Temp)
Maximum Moment about Major Axis	**37400 KNm**	(Dead + SIDL + EqX + Moving)
Maximum Moment about Minor Axis	**34262 KNm**	(Dead + SIDL + EqY + Moving)

Table 5.2: Forces for Pylon above Deck

5.4.3 Pylon above Deck (5 m)

Design Forces:

Forces	Value	Load Case
Maximum Axial Force	**25983 kN**	(Dead + SIDL + Wind + Temp)
Maximum Moment about Major Axis	**23015 kNm**	(Dead + SIDL + EqX + Moving)
Maximum Moment about Minor Axis	**28783 kNm**	(Dead + SIDL + EqY + Moving)

Table 5.3: Forces for Pylon above Deck

5.4.4 Pylon above Deck (1 m from 18m to 19 m)

Design Forces

Force	Value	Load Case
Maximum Axial Force	**23279 KN**	(Dead + SIDL + Wind + Temp)
Maximum Moment about Major Axis	**13645KNm**	(Dead + SIDL + EqX + Moving)
Maximum Moment about Minor Axis	**8027 KNm**	(Dead + SIDL + EqY + Moving)

Table 5.4: Forces for Pylon above Deck

5.4.5 Pylon above Deck (1 m from 19m to 20 m)

Design Forces

Force	Value	Load Case
Maximum Axial Force	**21587 KN**	(Dead + SIDL + Wind + Temp)
Maximum Moment about Major Axis	**14623 KNm**	(Dead + SIDL + EqX + Moving)
Maximum Moment about Minor Axis	**10188 KNm**	(Dead + SIDL + EqY + Moving)

Table 5.5: Forces for Pylon above Deck

5.4.6 Pylon above Deck (1 m from 20m to 21 m)

Design Forces

Force	Value	Load Case
Maximum Axial Force	**20352 kN**	(Dead + SIDL + Wind + Temp)
Maximum Moment about Major Axis	**15689 kNm**	(Dead + SIDL + EqX + Moving)
Maximum Moment about Minor Axis	**12877 KNm**	(Dead + SIDL + EqY + Moving)

Table 5.6: Forces for Pylon above Deck

5.4.7 Pylon above Deck (1 m from 21m to 22 m)

Design Forces

Force	Value	Load Case
Maximum Axial Force	**19165 kN**	(Dead + SIDL + Wind + Temp)
Maximum Moment about Major Axis	**16748 kNm**	(Dead + SIDL + EqX + Moving)
Maximum Moment about Minor Axis	**15929 kNm**	(Dead + SIDL + EqY + Moving)

Table 5.7: Forces for Pylon above Deck

5.4.8 Pylon above Deck (1 m from 22m to 23 m)

Design Forces

Force	Value	Load Case
Maximum Axial Force	**17924 kN**	(Dead + SIDL + Wind + Temp)
Maximum Moment about Major Axis	**17393 kNm**	(Dead + SIDL + EqX + Moving)
Maximum Moment about Minor Axis	**19087 kNm**	(Dead + SIDL + EqY + Moving)

Table 5.8: Forces for Pylon above Deck

5.4.9 Pylon above Deck (1 m from 23m to 24 m)

Design Forces

Force	Value	Load Case
Maximum Axial Force	**16698 kN**	(Dead + SIDL + Wind + Temp)
Maximum Moment about Major Axis	**17437kNm**	(Dead + SIDL + EqX + Moving)
Maximum Moment about Minor Axis	**22116 kNm**	(Dead + SIDL + EqY + Moving)

Table 5.9: Forces for Pylon above Deck

5.4.10 Pylon above Deck (1 m from 24m to 25 m)

Design Forces

Force	Value	Load Case
Maximum Axial Force	**15510 kN**	(Dead + SIDL + Wind + Temp)
Maximum Moment about Major Axis	**17455 kNm**	(Dead + SIDL + EqX + Moving)
Maximum Moment about Minor Axis	**24839 kNm**	(Dead + SIDL + EqY + Moving)

Table 5.10: Forces for Pylon above Deck

5.4.11 Pylon above Deck (1 m from 25m to 26 m)

Design Forces

Force	Value	Load Case
Maximum Axial Force	**14224 kN**	(Dead + SIDL + Wind + Temp)
Maximum Moment about Major Axis	**16759 kNm**	(Dead + SIDL + EqX + Moving)
Maximum Moment about Minor Axis	**27108 kNm**	(Dead + SIDL + EqY + Moving)

Table 5.11: Forces for Pylon above Deck

5.4.12 Pylon above Deck (1 m from 26m to 27 m)

Design Forces

Force	Value	Load Case
Maximum Axial Force	**12857 kN**	(Dead + SIDL + Wind + Temp)
Maximum Moment about Major Axis	**15235 KNm**	(Dead + SIDL + EqX + Moving)
Maximum Moment about Minor Axis	**28820 kNm**	(Dead + SIDL + EqY + Moving)

Table 5.12: Forces for Pylon above Deck

5.4.13 Pylon above Deck (1 m from 27m to 28 m)

Design Forces

Force	Value	Load Case
Maximum Axial Force	**11393 kN**	(Dead + SIDL + Wind + Temp)
Maximum Moment about Major Axis	**12820kNm**	(Dead + SIDL + EqX + Moving)
Maximum Moment about Minor Axis	**29099 kNm**	(Dead + SIDL + EqY + Moving)

Table 5.13: Forces for Pylon above Deck

5.4.14 Pylon above Deck (1 m from 28m to 29 m)

Design Forces

Force	Value	Load Case
Maximum Axial Force	**9721 kN**	(Dead + SIDL + Wind + Temp)
Maximum Moment about Major Axis	**9629 kNm**	(Dead + SIDL + EqX + Moving)
Maximum Moment about Minor Axis	**29358 kNm**	(Dead + SIDL + EqY + Moving)

Table 5.14: Forces for Pylon above Deck

5.4.15 Pylon above Deck (1 m from 29m to 30 m)

Design Forces

Force	Value	Load Case
Maximum Axial Force	**7902 kN**	(Dead + SIDL + Wind + Temp)
Maximum Moment about Major Axis	**6832 kNm**	(Dead + SIDL + EqX + Moving)
Maximum Moment about Minor Axis	**29578 kNm**	(Dead + SIDL + EqY + Moving)

Table 5.15: Forces for Pylon above Deck

5.4.16 Pylon above Deck (1 m from 30m to 31 m)

Design Forces

Force	Value	Load Case
Maximum Axial Force	**5752 kN**	(Dead + SIDL + Wind + Temp)
Maximum Moment about Major Axis	**2968 kNm**	(Dead + SIDL + EqX + Moving)
Maximum Moment about Minor Axis	**29770 kNm**	(Dead + SIDL + EqY + Moving)

Table 5.16: Forces for Pylon above Deck

5.4.17 Pylon above Deck (1 m from 31m to 32 m)

Design Forces

Force	Value	Load Case
Maximum Axial Force	**3119 kN**	(Dead + SIDL + Wind + Temp)
Maximum Moment about Major Axis	**1490 kNm**	(Dead + SIDL + EqX + Moving)
Maximum Moment about Minor Axis	**28689 KNm**	(Dead + SIDL + EqY + Moving)

Table 5.17: Forces for Pylon above Deck

5.5 DESIGN OF PYLON BELOW DECK (9.9 M HEIGHT)

Width of Column b	=	**2500**	mm
Depth of the Column D	=	**3500**	mm
Grade Of Concrete	=	**40**	N/mm2
Grade Of Steel	=	**415**	N/mm2
Factrode Axial Load	=	**36589**	kN
Factrode Moment about Major Axis	=	**46087**	kNm
Factrode Moment about Minor Axis	=	**39523**	kNm
Height of the Column	=	**9900**	mm
Efective Length about Major Axis (L_{ex})	=	**11880**	mm
Efective Length about Minor Axis (L_{ey})	=	**11880**	mm
Cove To Reinforcement	=	**100**	mm

Check For Slenderness:

About Major Axis : L_{ex}/D = **3.39** < 12
Column is Short About Major Axis

About Minor Axis : L_{ey}/b = **4.7** < 12
Cloumn is Short About Minor Axis

About Major Axis;
Here;

d'/D = 0.02

$$\left[\frac{P_u}{f_{ck} * b * D}\right] = 0.105$$

Assume Percentage of Steel

p = 1.75 %

So,

p/f_{ck} = **0.04**

For;

$$\left[\frac{P_u}{f_{ck} * b * D}\right] = \mathbf{0.105}$$

p/f_{ck} = **0.044**

From Appropriate Chart of SP-16, get $\left[M_u\right]$

the value of $\overline{f_{ck}\ b\ D^2}$

Here From Chart No. **43** of SP-16 we get

$$\left[\frac{M_u}{f_{ck}\ b\ D^2}\right] = \mathbf{0.0.9}$$

Therefore; $\mathbf{M_{ux1}}$ = **110250** **kNm** **46087**
Ok

About Minor Axis;

Here;

$$d'/b = 0.040$$

$$\left[\frac{P_u}{f_{ck} * b * D}\right] = 0.1$$

Assume Percentage of Steel;

$$p = 1.75 \quad \%$$

So,

$$p/_{fck} = \mathbf{0.04}$$

Therefore, for :

$$\left[\frac{P_u}{f_{ck} * b * D}\right] = \mathbf{0.10}$$

$$d'/b = \mathbf{0.04}$$

$$p/_{fck} = \mathbf{0.04375}$$

From Appropriate Chart of SP-16 get the value of $\left[\frac{M_u}{f_{ck}\ b^2\ D}\right]$

From Chart No. **43** of SP-16

$$\left[\frac{M_u}{f_{ck}\ b^2\ D}\right] = \mathbf{0.09}$$

Therefore;

M_{uy1}	=	**78750**	kNm	**39523**	kNm
		Ok			

Pure Axial Load Capacity:

Puz **=** **(0.45 * Fck * Ac)+(0.75 * Fy * As)**

Ac	=	8596875	mm^2
A_s	=	157500	mm^2
Puz	=	**202403.9**	**KN**
Pu/Puz	=	**0.18**	
α_n	=	1.0	

Pu/Puz	$\boldsymbol{\alpha_n}$
0 to 0.2	1
0.4	1.33
0.6	1.67
0.8 or more	2

Check:

$$\left\{\frac{Mux}{Mux1}\right\} + \left\{\frac{Muy}{Muy1}\right\}$$

Should not be greater than 1

Here;

0.9199

Ok

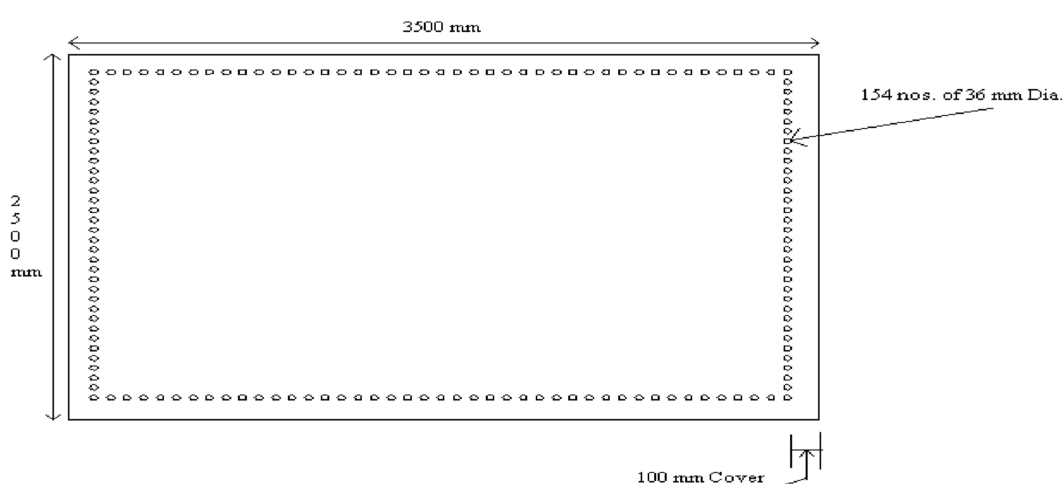


Fig 5.1: Section of Pylon at Base

5.6 RESULTS FOR DECK

Deck for the cable stayed bridge is provided as steel welded plate girder. Design forces are obtained from the SAP-2000 analysis. Maximum Shear Force and Maximum Moment obtained from SAP analysis is used for the Design of the section.

Design Forces

Forces	Values	Load Case
Maximum Shear Force	**2131 KN**	(Dead + SIDL +EQX + Moving)
Maximum Moment	**16668 KNm**	(Dead + SIDL + EQY + Moving)

Table 5.18: Forces for Deck

5.7 DESIGN OF PLATE GIRDER

Total Length of Deck	288	m		
Fy	250	N/mm2		
Bending Moment (KN m)	16688	kN m		
Effective Span	2.25	m		
Shear Force (KN)	2131	kN		
Thickness Of Web (mm)	14	mm		
Permissible Bending Stress (N/mm2)	165	N/mm2		
Permissible average shear stress (N/mm2)	100	N/mm2		
Depth of Web Required (mm)	1.1 $\sqrt{\frac{Mu}{bt\ \sigma t}}$	2956.5		**3000**
Average shear stress in Web		50.7381		
Area requried for single flange (mm2)	28463.13		Say	28500
Area of each flange required	28463.13		Say	28500

Thickness of Flange Plates	55	mm
Width of Flange plate	550	mm
Area Of Flange Plate	30250	mm2

Total Area Of Flanges Provided 60500 mm2

Area Provided Is Ok

Flange Outstand Should not be greater than
12 X Thickness of Flange Plate 660 mm

Flange Outstand 268 mm

Less than Maxiamum Allowable, so OK

Therefore Provide a Plate girder With:

Web =	**3000**	**X**	**14**	**mm**
Each Flange =	**550**	**X**	**55**	**mm**

Check for Bending Stresses

Moment of Inertia

Moment of Inertia of Web	bD3/12	3.15E+10	**mm^4**
Moment of Inertia of Flanges	bd3/12	1.41E+11	**mm^4**
Total Moment of Inertia of the section		**1.73E+11**	**mm^4**

Maximum Bending stress :

150.2794339 **N/mm^2** **Section is OK**

CONNECTION

Horizontal Shear **V*A*Y/ Ixx**

V 2131 kN

A*Y	**4.62E+07**	**mm^3**
Ixx	**1.73E+11**	**mm^4**

Horizontal Shear/mm	V*A*Y/ Ixx	=	570.24	N/mm
Size Of Weld	Welding is done on both sides of the web			
Minimum size of Weld			10	mm
Maximum size of Weld			45	mm

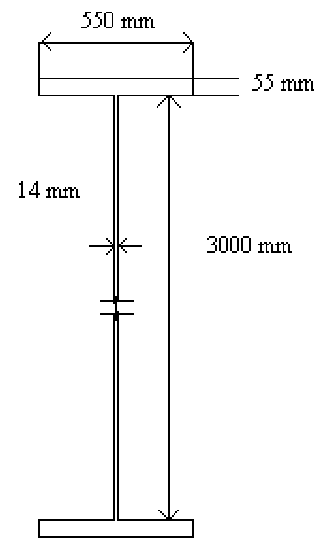


Fig 5.2: Plate Girder Section

So provide size of weld between 10mm to 45 mm

Let us provide weld of size s = 10 mm

The effective weld length is 4s = 40

Therefore provide 40 mm long intermittent filled weld

Pitch of the weld = Strength of weld on both faces of web/Horizontal shear per mm

108.025379 mm

The permissible clear spacing between welds should not be more than 12t or 200 mm whichever is less.

So provide 40 mm long fillet welds at a pitch of 144 mm.

Bearing Stiffener

Maximum Shear Force	=	2131	KN	
Allowable bearing stress	=	0.75*Fy	187.5	N/mm^2

Bearing Area Required	=	**11365.33333**	**mm^2**	
Let us try two plates 200 mm wide as the stiffner		200	mm	
Thickness of plate = 28.41333333	Round to higher even No.	30	mm	
Outstand should not be more than 12t	=	360	mm	
Bearing Area Provided	=	12000	**mm^2**	**OK**

Which is Sufficient

Area of Stiffener	=	**23760**	**mm^2**
Ixx	=	**1.77E+08**	**mm^4**
Radius of Gyration	=	**86.405005**	**mm**
Effective Length (0.7*Depth)	=	**2100**	**mm**
l/r	=	**24.30**	

From IS: 800- 1984 Table 5.1, for:	l/r_y	**24.30**	
	F_y	**250**	
Permissible stress in Compression	=	**144**	N/mm2

Safe Load(Area*Permissible stress)	**3421.44**	**kN**	**OK**
So Provide stiffeners of	**200**	**X**	**30**

Connection

Let us Provide 6mm size intermittent fillet welds.

Strength of the weld	0.7*6*110	**462**	**N/mm**
Required Strength of Weld/mm	Shear Force/4*depth		**177.58**

The Length of intermittent fillet weld is taken as 10t = 10*(Thickness of Bearing Stiffener)

Therefore Provide fillet weld of 200 mm

C/C Spacing of weld (Weld Length* Strength of Weld)/Required strength of weld

520.3191 > 300mm

Hence Provide 200 mm long fillet welds at a spacing of 300 mm.

Intermediate Stiffeners

For an unstiffened web the minimum required thickness is as below:

1) $\mathbf{t_{w,min}=d1(T_{va})^{1/2}/816}$

$T_{va}=$ 50.738095 N/mm2

$t_{w,min}=$ **26.19** mm

2) $\mathbf{t_{w,min}=(d1(f_y)^{1/2})/1344}$

35.29 mm

3) $\mathbf{t_{w,min}=d1/85}$

35.29 mm

The web thickness provided is quite less than required if it is to be unstiffened. Therefore, intermediate web sitffeners will be requred.

Vertical stiffeners are provided if the web thickness as calculated below is mor than the thicknes provided.

d2	=	**2500**	mm
(i) $\mathbf{t_{w,min}=d2*(f_y)^{1/2}/3200}$	=	**12.35**	mm
(ii) $\mathbf{t_{w,min}=d2/200}$	=	**12.5**	mm

Vertical stiffener not required

The web thickness provided is 14 mm which is less than 12.5 mm. Hence vertical stiffeners will not be provided.

The horizontal stiffener will be required at a distance of 2/5 of the distance from the

is more than that provided.

(i) **$t_{w,min}$=d2*(f)$^{1/2}$/4000** = **9.88** mm

(ii) **$t_{w,min}$= d2/250** = **10** mm

Horizontal stiffener not required

As the web thickness provided is more than the calculated min. required, horizontal stiffener will not be requried.

Horizontal Stiffener not required

Horizontal stiffener will be required at the N.A. if the web thickness calculated below
is more than the provided.

(i) **$t_{w,min}$=d2*(f_y)$^{1/2}$/6400** = **6.18** mm

(ii) $t_{w,min}$=d2/400 = **6.25** mm

As thickness of web provided is more than the calculated above, second horizontal stiffener is not required.

5.8 DESIGN OF PILE GROUP

No. of Piles	=	**16**	
Max X Distance	=	**4.5**	**m**
Max Y Distance	=	**4.5**	**m**
Sum(n*x^2)	=	**162**	**m^2**
Sum(n*y^2)	=	**162**	**m^2**

Load Combination	P (kN)	Major Axis (kNm)	Minor Axis (kNm)	Due to P (kN)	Due to Major (kN)	Due to Minor (kN)	Pile Load Max (kN)	Pile Load Min (kN)
Comb1	28700	23150	129	1793.75	643.06	3.58	2440.39	1147.11
Comb2	28877	46087	1185	1804.81	1280.19	32.92	3117.92	491.70
Comb3	19279	14677	39523	1204.94	407.69	1097.86	2710.49	-300.62
Comb4	27505	42365	1181	1719.06	1176.81	32.81	2928.67	509.45

Comb5	17907	10069	29365	1119.19	279.69	815.69	2214.58	23.80
Comb6	36589	23602	969	2286.81	655.61	26.92	2969.34	1604.28
Comb7	27632	18949	1000	1727	526.36	27.78	2281.14	1172.86

Max.	**3117.92**	**kN**
Min	**-300.62**	**kN**

Deisgn Pile for maximum load capacity of 3100 kN

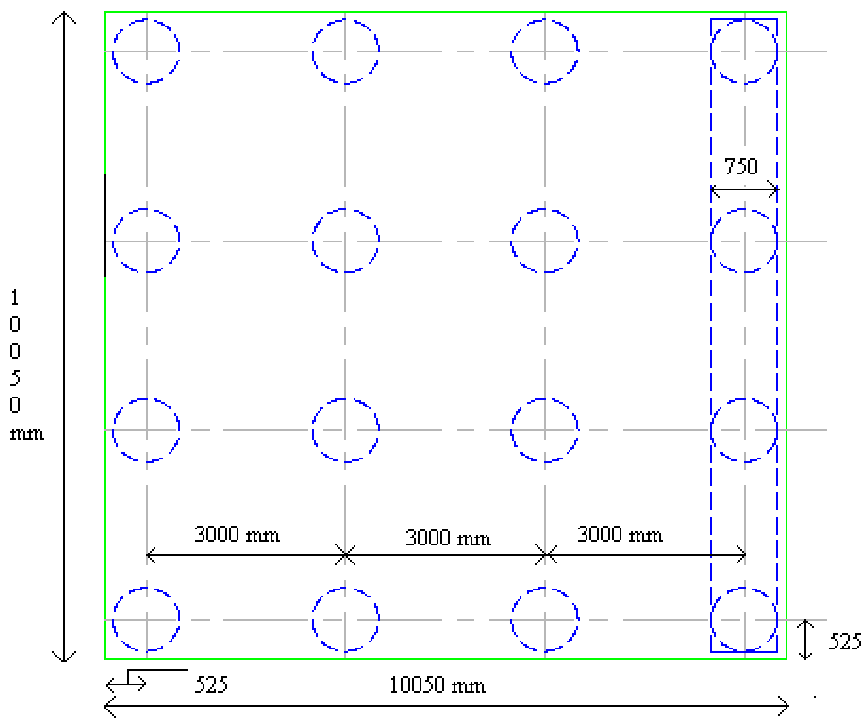


Fig 5.3: Pile Group

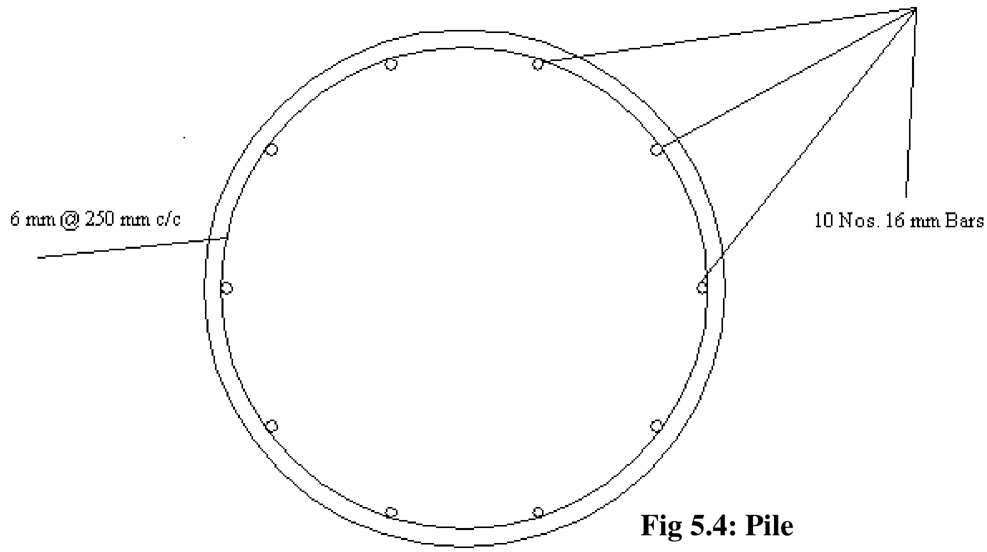


Fig 5.4: Pile

Load	=	**3117.92**	**kN**	$\mathbf{f_{ck}}$	**415**
				$\mathbf{f_y}$	**35**
Assume 1% steel	p=	1	%		

Then,

A_{sc} = 0.01 Ag

A_c = 0.990 Ag

$$P_u = 0.4\, f_{ck} * A_c + 0.67\, f_y * A_{sc}$$

Now,

$$3.12E+06 = 0.4 * f_{ck} * 0.99\, A_g + 0.67 * f_y * 0.01\, A_g$$

$$3.12E+06 = 13.86\, A_g + 2.7805\, A_g$$

$$3.12E+06 = 16.6405\, A_g$$

$$A_g = 187369.59\ mm^2$$

D is the Diameter of Pile

$$\text{Area} = (Pi/4)*D^2$$

D = 488.433 mm

So, Provide the Pile of 750 mm Diameter

Steel;

Area of Steel	=	1	%
A_{sc}	=	**1873.7**	**mm^2**

Provide 10 - 16 mm Bars

A_{sc} Provided	=	**2010**	**mm^2**

5.9 Design of Pile Cap

Diameter of pile	=	0.75	m	Area of Pile
Length of Pile	=	10	m	
Density of Concrete	=	25	kN/m2	
No. of Piles	=	16		
Dimension of Pile Cap				
Length of Pile Cap	=	10.05	m	
Width of Pile Cap	=	10.05	m	
Thickness Of Pile Cap	=	2	m	

Self Weight of Pile	=	110.4466	kN
Self Weight of Pile Cap	=	5050.125	kN
Hence load per Pile	=	315.6328	kN
Vertical load Coming on Pile Cap	=	36523	kN

Hence max reaction from pile excluding its self Weight
2604.9206 kN

Hence Max B.M. in the pile cap from max loaded pile is

8752.5331 kNm

Hence moment per meter width = 870.8988 kNm

From the Equation

Pt = 0.006

Ast = 0.0061 m2

Ast = **6060.15** mm2

so Provide 32 mm bars at 160 c/c at bottom and 16 mm bars @ 160 mm c/c at top

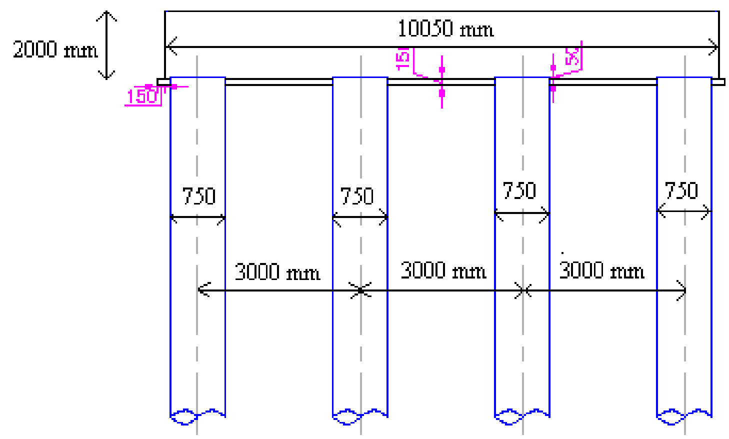


Fig 5.5:Pile Cap

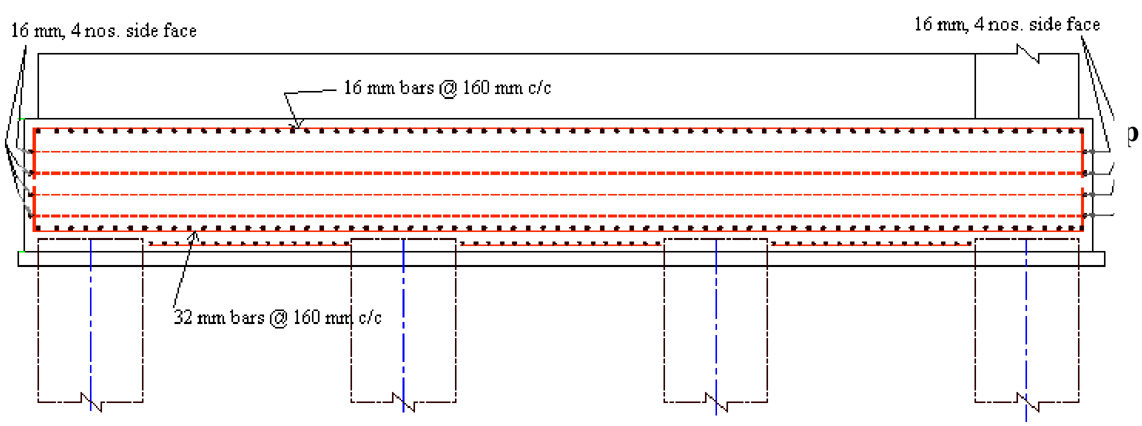


Fig 5.6: Pile Cap Reinforcement

CHAPTER 6

QUANTITY AND ABSTRACT OF COMPONENTS

6.1 QUANTITIES OF COMPONENTS

6.2 QUANTITY OF CABLES

Density of 7 mm Strands = 0.3021 Kg/m

Cable	Length	No. of 7 mm Wires	Quantity (Kg)	Total No. of Similar Cables
C1	71.55	122	10548.24	4
C2	67.4	132	10750.89	4
C3	63.26	116	8867.43	4
C4	59.14	70	5002.53	4
C5	55.05	70	4656.57	4
C6	50.98	54	3326.63	4
C7	46.95	46	2609.78	4
C8	42.97	42	2180.85	4
C9	39.04	36	1698.33	4
C10	35.2	34	1446.21	4
C11	31.46	30	1140.49	4
C12	27.88	28	943.33	4
C13	24.5	28	828.96	4
C14	21.49	30	779.06	4
C15	18.94	70	1602.10	4
C16	18.94	70	1602.10	4
C17	21.9	66	1746.62	4
C18	25.55	38	1173.24	4
C19	29.62	40	1431.71	4
C20	33.98	44	1806.70	4
C21	38.52	48	2234.28	4
C22	43.18	52	2713.29	4
C23	47.92	56	3242.77	4
C24	52.75	60	3824.59	4
C25	57.61	60	4176.96	4
C26	62.5	60	4531.50	4
C27	67.45	66	5379.43	4
C28	72.4	66	5774.22	4
C29	77.38	60	5610.36	4
C30	82	116	11494.30	4

Total Quantity of Cables	**113123.47**	Kg
	113.1234718	Tonnes

6.3 QUANTITY FOR PYLON

Quantity of Steel and Concrete for Pylon

Pylon Height (mts)	Elevation	Area Of Steel (m2)	Volume of Steel (m3)	Area of Concrete (m2)	Volume of Concrete (m3)
9.9	(-10m-0m)	0.153	1.515	8.750	86.625
13.42	(0m-13m)	0.157	2.107	8.750	117.425
5	(13m-18m)	0.125	0.625	6.250	31.25
1	18m-19m	0.060	0.060	4.000	4
1	19m-20m	0.060	0.060	4.000	4
1	20m-21m	0.060	0.060	4.000	4
1	21m-22m	0.060	0.060	4.000	4
1	22m-23m	0.060	0.060	4.000	4
1	23m-24m	0.070	0.070	4.000	4
1	24m-25m	0.070	0.070	4.000	4
1	25m-26m	0.070	0.070	4.000	4
1	26m-27m	0.070	0.070	4.000	4
1	27m-28m	0.110	0.110	4.000	4
1	28m-29m	0.110	0.110	4.000	4
1	29m-30m	0.110	0.110	4.000	4
1	30m-31m	0.110	0.110	4.000	4
1	31m-32m	0.110	0.110	4.000	4
42 m			**5.37664**		**291.3**

Total Volume of Steel for Whole Pylon =	5.37664	
Density of Steel	78.25	kN/m^3
Quantity for One Pylon:		
Total Weight of Steel	420.72208	kN
	42.072208	**Tonne**
Total Volume of Concrete	**291.3**	**m^3**
Quantity for all 4 Pylons Super structure as well as sub structure:		
Steel	**168.288832**	**Tonne**
Concrete	**1165.2**	**m^3**

6.4 QUANTITY OF STEEL FOR PLATE GIRDER

Area of Plate Girder Provided:	=	**97000**	**mm^2**	i.e	**0.097**	**m^2**
Area of Bearing Stiffener	=	**12000**	**mm^2**	i.e.	**0.012**	**m^2**
					0.109	**m^2**
Volume of steel Required	=	Area x Length		31.392	m^3	
Density of Steel	=	**78**	kN/m^3			
Weight of Steel Required	=	**2448.576**	kN	**244.8576**	**Tonne**	
Total Weight of Plate Girder	=	**4897.152**		**489.7152**	**Tonne**	

Quantity of Steel for Floor Beam and Rib Beam

Area of Floor Beam	=	20000	mm^2	0.02	m^2
Area of Rib Beam	=	1750	mm^2	0.00175	m^2
Volume of Steel for Floor Beam	=	**33.75**	m^3		
Volume of Steel for Rib Beam	=	**1.512**	m^3		
		35.262	m^3		

Weight of steel for Rib Beam and Floor Beam 2750.436 kN

275.04 **Tonne**

Thus total weight of steel required for deck = **764.76** **Tonne**

6.5 QUANTITY FOR PILE GROUP

Density of Steel	=	**78.25**	**kN/m^3**		
Area of Steel Per Pile	=	2110	mm^2	**0.00211**	**m^2**
Area of Concrete per Pile	=	439676.47	mm^2	**0.439676**	**m^2**
Length of Pile	=	4000	mm^2	**4**	**m**
Volume of Steel Per Pile	=	8440000	mm^3	**0.00844**	**m^3**
Volume of Concrete per Pile		1758705868	mm^3	**1.76**	**m^3**

Total no. of Piles in a group	**16**	
So, Quantity of steel for Pile Group	**0.13504**	**m^3**
So, Quantity of Concrete for Pile Group	**28.1393**	**m^3**

There are in Total 4 Pile Groups
Therefore, total quantity for all Pile Groups

Total Quantity of Steel	**0.54016**	**m^3**
Total Quantity of Concrete	**112.55**	**m^3**
Weight of steel required:	**42.26752**	**kN**
	4.226752	**Tonne**

6.6 Quantity for Pile Cap

Length of pile cap	=	10.05	m
Width of Pile Cap	=	10.05	m
Depth of pile Cap	=	2	m
Density of Steel	=	78	kN/m^3
Volume of Concrete	=	**202.005**	m^3
	=	**808**	$\mathbf{m^3}$
For all 4 Caps			
Area of Steel per meter	=	6060	mm^2
Volume of Steel for Pile Cap	=	60903	mm^3
	=	0.061	m^3
Weight of Steel Required	=	**4.7504**	kN
		0.475	Tonne
Steel for all 4 Caps	=	**1.9**	Tonne

6.7 ABSTRACTS OF CABLE-STAYED BRIDGE

Rate (Rs.)	Item	Quantity	Unit	Amount (Rs.)
	Pile Foundation			
	Earthwork in excavation for foundation for Structures in all types of strata for all depths including Dressing of bottom and sides of trenches, stacking excavated stuff clear from the edges of the excavation Including disposal of surplus soil.			
100	For Pylon Pile Cap	**808.02**	Cubic Meter	**80802**
	Empty Boring for required diameter for bo red reinforced cement concrete cast in situ piles including withdrawal of shell, removal of earth including all lift and leads.			

442	For Pylon Piles	144	Running mts	63648
1767	Providing and laying in situ cement concrete M-15 grade of trap metal for leveling course below pile cap.	60.6	Cubic mts	107080
9359	Boring and providing M-35 cast in situ bored piles each of load capacity as designed and of 750 mm diameter.	480	Running mts	4492320
2512	Providing and laying in situ cement concrete M-35 grade of trap metal for R.C.C. Pile cap.	808	Cubic mts	2029696
	Providing and fixing in position HYSD bar reinforcement of various diameter.			
33000	For Pylon Piles	45	MetricTonne	1485000
	For Pylon pile Cap	32	MetricTonne	1056000
28392	Providing, placing and driving in position 6 mm thick permanent MS Liner up to required depth with 12 mm MS cutting edge of 0.5 M length at bottom.	80.64	MetricTonne	2289531

Sub Structure

6800	Providing and laying in situ cement concrete M-40 of rap metal for substructure up to deck level including supply of all material with all leads and lifts as per approved detailed design and drawing including necessary steel shuttering, steel centering, compaction by vibrating, finishing and curing.	346.48	Cubic Meter	2356064
3123	Providing and laying in situ cement concrete M-35 grade of trap metal for substructure(pier cap and Pedestal) including supply of all material with all leads and lifts as per approved detailed design and drawing including necessary steel shuttering, steel centering, compaction by vibrating, finishing and curing.	16	Cubic meter	49968

33000	Providing and fixing in position HYSD/MS bar reinforcement of various diameter for substructure as per detailed design and drawing.	**46.8**	Metric Tonne	**1544400**
	Super Structure			
35000	Providing and laying steel for plate girder, floor beam and rib beam in deck superstructure as per approved design and drawing.	**770**	Metric tonne	**26950000**
6800	Providing and laying concrete of M-40 grade for PYLON superstructure above deck.	**818**	Cubic meter	**5562400**
33000	Providing and laying in position HYSD/MS bar Reinforcement of various Diameters.	**200**	Metric tonne	**6600000**
525000	Manufacture, supply, erection and stressing of stay cables with 3 layers of corrosion protection, made of high tensile/ high fatigue resistant parallel wires with galvanization and HDPE sheathing filled with corrosion inhabiting grease or wax inside and with positive anchorage using bottom headed system and shop fabricated under controlled conditions. Rate including cost of all accessories like Bering plates, DINA anchorages with epoxy compound, lock nuts, neoprene vibration dampers, guide trumpets and cones, shrink sleeves, end caps etc. including cost of plant and machinery equipments, transportation of prefabricated stay cables to site in special bobbins. Cost of erection using special equipments, winches, deviators etc, including stressing of cables to the design forces, using high capacity hydraulic jacks and power packs and accessories, recording of stressing data etc. and all other incidentals necessary to construct the stay cables in accordance with drawings and technical specification.	**113.12**	Metric Tonne	**59388000**

	Providing GI antivandalism pipes/stays trumpet pipes in bridge deck and pylon as per detailed design and drawings, with precision prefabrication at workshop including machining if required, testing, fixtures etc. transporting to site.			
3000	Providing and laying in situ cement concrete M-35 grade of trap metal for Crash Barrier.	**293**	Cubic Meter	**879000**
30358	Providing GI antivandalism pipes/stays trumpet pipes in bridge deck and pylon as per detailed design and drawings, with precision prefabrication at workshop including machining if required, testing, fixtures etc. transporting to site.	**22**	Metric tonne	**667876**
13033	Providing and fixing sliding pot bearing with PTFE surface sliding on stainless steel to the true line and level and in the position as per drawing so as to impart full and even bearing on the seats and free movements/restraints as specified including all leads and lifts etc. complete and testing.	**8**	Nos.	**104264**
3618	Providing and laying Dense bituminous macadam 50 mm thick with 5%.	**194**	Cubic meter	**701892**
408	Providing and laying bitumen mastic wearing course of 25 mm thick over road pavement or bridge decks over bituminous macadam base or cement concrete based form a dense impermeable surface including supplying and conveyance of all materials, labor, prepare bitumen mastic, laying, jointing, finishing etc. completed as directed by engineer.	**3888**	Square meter	**1586304**
	Carrying out various tests on soil and rock samples as directed and to submit the detailed soil investigation report.		Lump Sum	**100000**
	Total Cost of Cable Stayed Bridge	**Rs.**	**118094245**	

The total cost of cable stayed bridge comes out to: Rs. **118094245**

Increase the total cost of bridge by 10% to account for sundry and miscellaneous items.

Cost of Cable Stayed Bridge: Rs. **129903669**

(12 Crores, 99 Lakhs, 3 Thousand, 6 Hundred and 69 rupees)

Length of bridge is **288** mts.

So, per meter running cost of cable stayed bridge comes out to Rs. **451054**

(4 Lakhs, 51 Thousand and 54 rupees)

6.8 Approximations In Abstract Of Cable Stayed Bridge

In the present study, rates of some fixed items like drainage spouts, traffic improvement scheme, drain water service line duct, cat eye, coloured, railing, brick masonry inspection chamber etc. are not included for simplicity. Total cost of cable stayed bridge presented over here is excluding these fixed items which in field are actually accounted for.

6.9 Proposed Road Over Bridge At Nagpur

A 3-Lane Cable Stayed road over bridge is proposed in Nagpur railway station, following is the data of the proposed cable stayed bridge:

Overall span of the bridge	200 mts
Cable arrangement	Semi-Harp type
Bridge Deck	Prestressed Deck Slab
Pylon Arrangement	Inverted Y Single Pylon
Height of Pylon	45 mts

6.9.1 Cost of the proposed cable stayed bridge at Nagpur:

Cost of Bridge at Nagpur excluding the cost of fixed items not considered in the present study:

Rs. 109105569/- (10 Crores, 91 Lakhs, 5 thousand, 5 Hundred and 69 rupees)

Length of the proposed cable stayed bridge: 200 mts

Therefore, per running meter cost of proposed bridge:

Rs. 545527/- (5 Lakhs, 45 Thousand, 5 Hundred and 27 rupees)

CHAPTER 7
Parametric Study

The behavior of cable stayed bridge is largely affected by various parameters. Some of the important parameters are i) Flexural rigidity of Pylon with hinged and fixed bases, ii) Effect of height of Pylon iii) No. of Cables iv) Side span to central span ratio v) Flexural rigidity of longitudinal girder.

The study has been carried out to investigate the effect of number of cables and ratio height of pylon to the central span on the behavior of radiating type cable stayed bridge.

The study has been carried out for bridges with 24, 30 and 36 cables per plane with side to main span ratio of 0.4 (i.e. 64 m Side Span) and height of tower to central span ratio of 0.15, 0.2 and 0.25.

The bridges have been analysed using SAP-2000 Software package.

The bridges have been treated as a two-dimensional structure for the purpose of analysis.

7.1 ASSUMPTIONS AND APPROXIMATIONS

→ All cables are fixed to the pylon and to the bridge girder at their points of attachment.

→ The cables are assumed to be perfectly flexible. i.e., the flexural stiffness of the cables can be neglected. Flexural rigidity of cables is very small as compared to that of girder and tower elements and hence neglected.

→ Effect of creep in steel is neglected.

→ 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. Podolny has shown that the effect of catenary action for moderate sag to span ratio is not large.

→ The effect of change in geometry, beam-column interaction of girder and pylon elements and the effect of wraping are neglected.

→ Only live load and dead load case are considered.

7.2 GEOMETRY OF BRIDGE

Central Span of the Bridge	160 Mts
Width of Bridge	13.5 Mts (2-Lanes)
Side Span to Main Span Ratio	0.40
Central Span to Height of Pylon ratio	0.15, 0.2 and 0.25
No. of Cables	24, 30 and 36
Cable Configuration	Radiating Type
Pylon Configuration	Double Plane system

Pylons are fixed at bases and roller supported at deck level.

The two side supports are hinged supported.

7.3 LOADS

Live Load	Class A Loading over Entire Span
Dead Load:	Self Weight Of all the components
SIDL	Weight Due to Crash Barrier and Ralling

7.3.1 Load Combinations

→ Load Case 1	Live load on entire Span
→ Load Case 2	Live load on side span only
→ Load Case 3	DL+LL+SIDL
→ Load Case 4	DL+ SIDL+LL (Whole Span)
→ Load Case 5	DL+SIDL+LL (Entire Span) + Temp Load

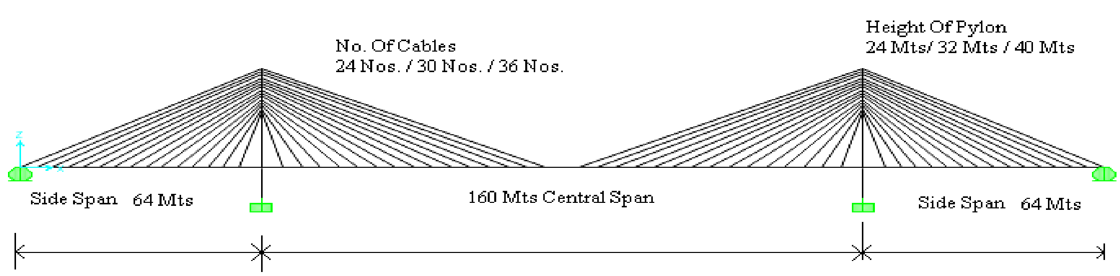


Fig 7.1: Bridge

Elevation

7.4 BRIDGES FOR STUDY

Side Span 64 mts:

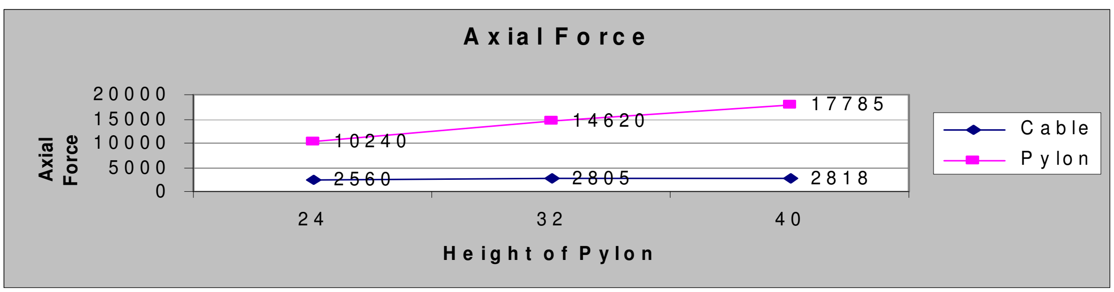
Height of the Pylon	No. of Cables
24 m	24
32 m	30
40 m	36

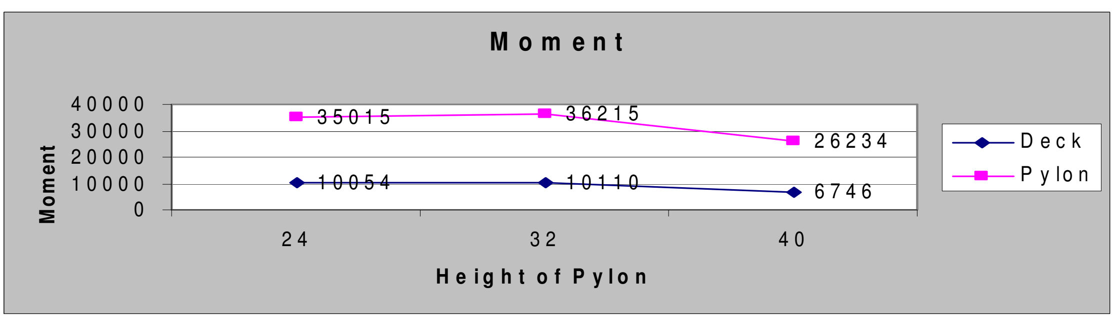
7.5 RESULTS OF PARAMETRIC STUDY

7.5.1 24 Cables System

Height of Pylon	Axial Force **In Cable**	Axial Force **In Pylon**	Central Deflection	Moment in **Deck**	Moment in **Pylon**
	(KN)	**(KN)**	**(mm)**	**(KNm)**	**(KNm)**
24 m	2560	10240	675	10054	35015
32 m	2805	14620	708	10110	36215
40 m	2818	17785	460	6746	26234

Table 7.1: Forces for 24 Cable System





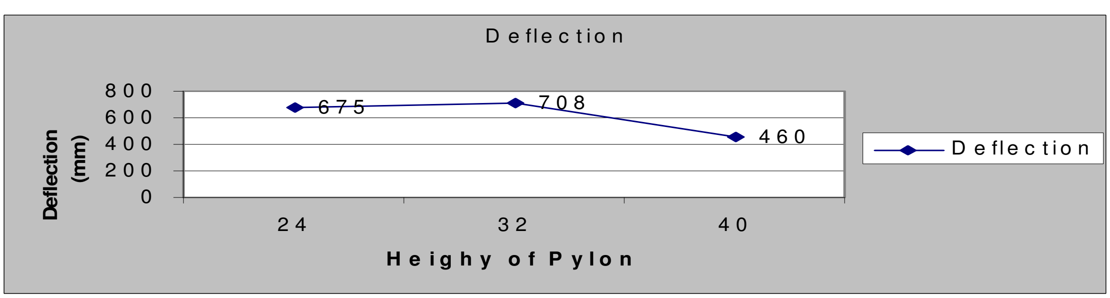
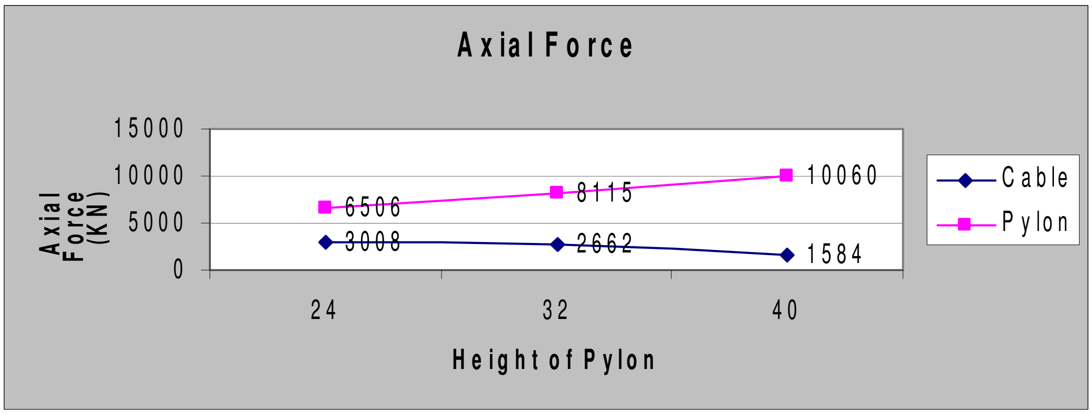


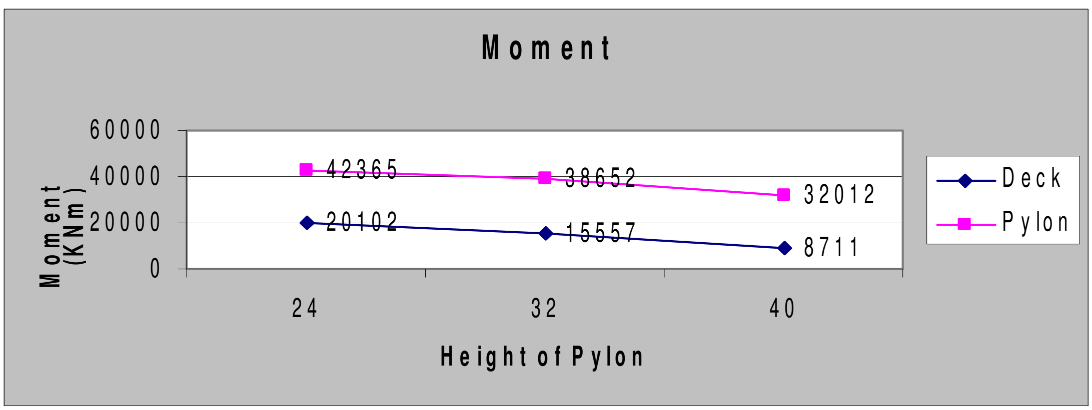
Fig 7.2: Comparison of Results for 24 Cable Bridge

7.5.2 30 Cables System

Height of Pylon	Axial Force **In Cable**	Axial Force **In Pylon**	Central Deflection	Moment in **Deck**	Moment in **Pylon**
	(KN)	**(KN)**	**(mm)**	**(KNm)**	**(KNm)**
24 m	3008	6506	1000	20102	42365
32 m	2662	8115	828	15557	38625
40 m	1584	10060	477	8711	32012

Table 7.2: Forces for 30 Cable System





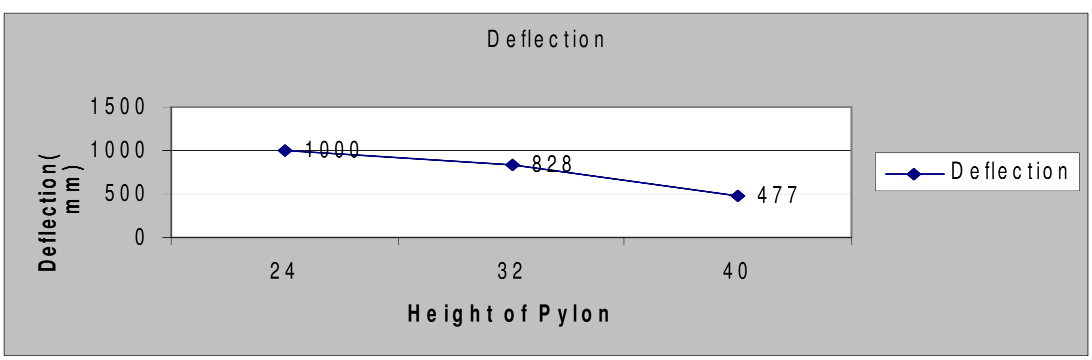
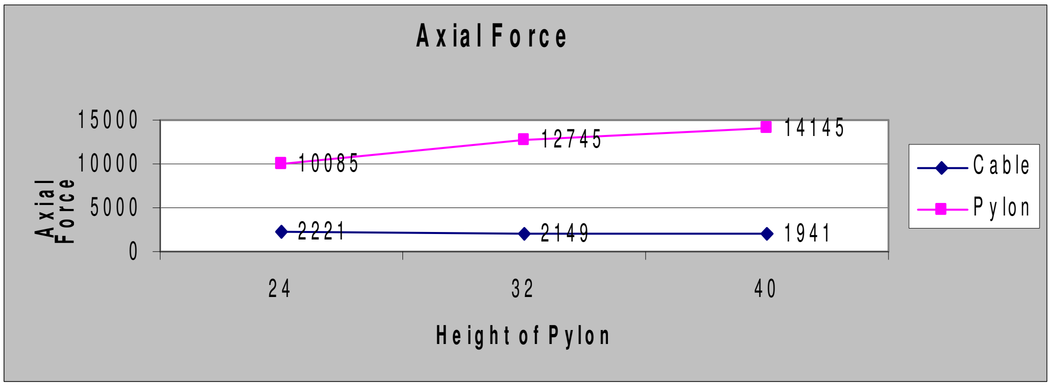


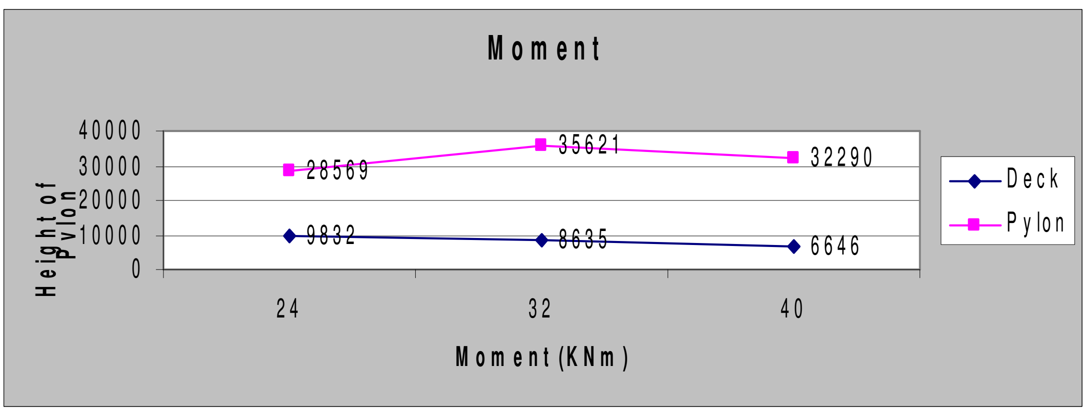
Fig 7.3: Comparison of Results for 30 Cable Bridge

7.5.3 36 Cables System

Height of Pylon	Axial Force **In Cable**	Axial Force **In Pylon**	Central Deflection	Moment in **Deck**	Moment in **Pylon**
	(KN)	**(KN)**	**(mm)**	**(KNm)**	**(KNm)**
24 m	2221	10085	593	9832	28589
32 m	2149	12745	499	8635	35621
40 m	1941	14245	400	6646	32290

Table 7.3: Forces for 40 Cable System





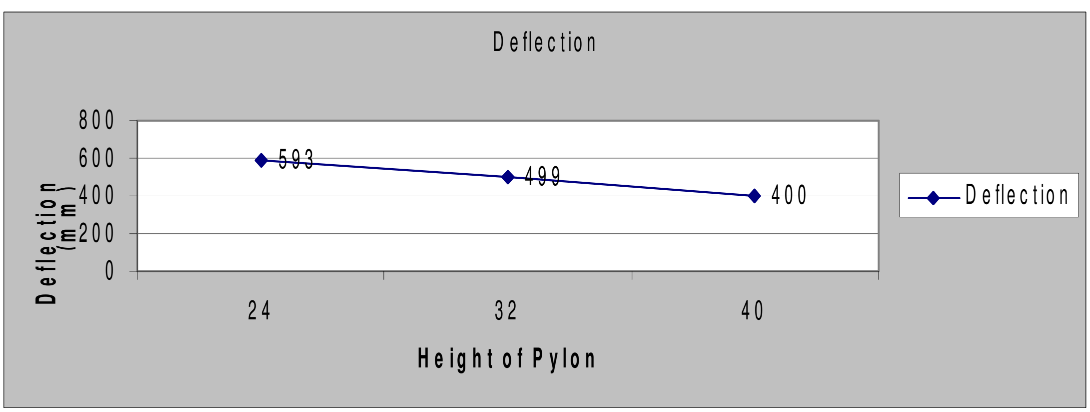


Fig 7.4: Comparison of Results for 36 Cable Bridge

7.6 COST OF INDIVIDUAL COMPONENTS FOR DIFFERENT BRIDGES

24 Cables System

Height of Pylon	Cost of Cables	Cost of Deck	Cost of Pylon
24 m	41743821	11612698	16895321
32 m	44292297	15419312	17956632
40 m	47391733	18545500	18547596

Table 7.4: Cost for 24 cable system

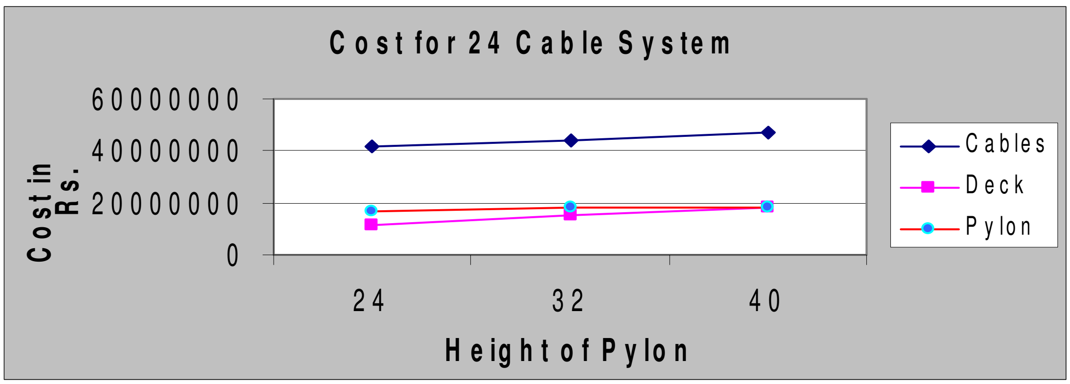


Fig 7.5 Comparison of cost of 24 cable bridge

30 Cables System

Height of Pylon	Cost of Cables	Cost of Deck	Cost of Pylon
24 m	49128912	17739589	14636258
32 m	44200931	18933678	18668951
40 m	36777875	16844429	15658732

Table 7.5: Cost for 30 Cable System

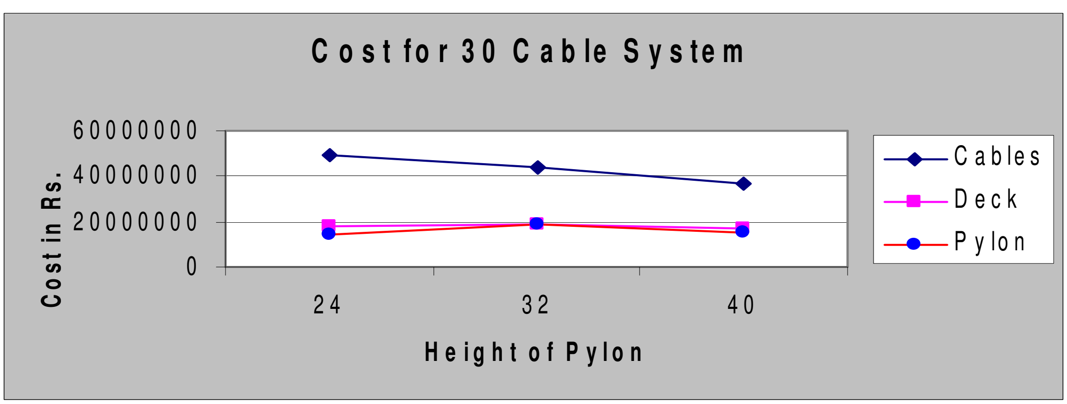


Fig 7.7 Comparison of cost of 30 cable bridge

36 Cables System

Height of Pylon	Cost of Cables	Cost of Deck	Cost of Pylon
24 m	49597066	17520947	15698365
32 m	40198195	17682688	18159742
40 m	41526764	19542707	20569832

Table 7.6: Cost for 36 Cable System

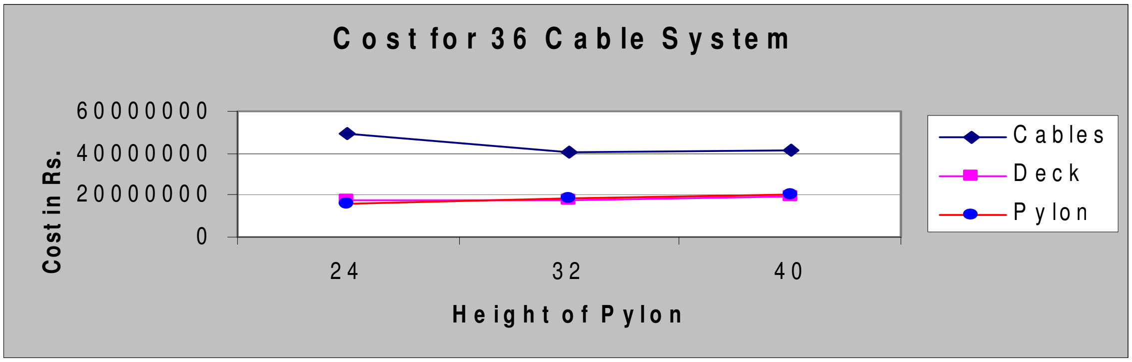


Fig 7.8 Comparison of cost of 36 cable bridge

7.6.1 Conclusion

- Cost of cables increases with the increase in height of pylon for 24 cable system while it decreases for 30 and 36 cable system.
- Cost of deck increases as the height of pylon is increased for all cable systems.
- Cost of pylon increases as the height of pylon is increased in height of Pylon.
- Maximum cost for Pylon, Cables and Deck is for 24 Cables, 40 mts height of Pylon.
- Minimum cost for Pylon, Cables and Deck is for 30 Cables, 40 mts height of Pylon.

CHAPTER 8
COST COMPARISION WITH TYPICAL BRIDGE

8.1 DATA OF THE EXISTING TYPICAL BRIDGE

Type of Bridge	:	Prestressed Concrete I-Girder Bridge
Width of the Bridge	:	12.5 mts (Two Lanes)
Span of the Bridge	:	40 mts
Type of Foundation	:	Pile Foundation

The bridge is Road Over Bridge at Palanpur on BG track and is proposed to be complete in Aug. 2005.

8.2 COST OF THE BRIDGE

The cost of the typical bridge taken for the study as per present rates of materials and labor is as under:

Total cost of the Bridge : **Rs. 17950000/-**

The span of this bridge is 40 mts, therefore per running meter cost of the bridge comes out to;

Total cost of Bridge / Span of the Bridge

Per running meter cost of bridge is : **Rs. 448750/-**

(Source: - Pankaj Patel & Associates, Ahmedabad)

8.3 COST OF CABLE STAYED BRIDGE

The cable stayed bridge taken for the present study is 288 mts of span. The foundation is pile foundation. The overall cost of the bridge taken for the study comes out to:

Total cost of Cable-Stayed Bridge : **Rs. 129903669/-**

The span of the bridge is 288 meters, therefore per running meter cost of the bridge comes out to:

Per running meter cost of bridge is : **Rs. 451054/-**

CHAPTER 9

CONCLUSION AND FUTURE SCOPE OF WORK

- Per running meter cost of Cable Stayed Bridge is not uneconomical as compared to the conventional bridges.
- A Cable Stayed option can be easily adopted if the situation demands it and the cost of cable stayed bridge is competitive when compared with the typical bridges.
- The economy in cost of Cable Stayed Bridge is attributed to less number of sub structures.
- Cost of steel plate girder is in accordance with any other steel plate girder bridge because the effective span of the deck is reduced by cables attached which acts like supports.
- Cost of cables comprises major part of the cost of bridge.
- Higher cost of Cable Stays is neutralized by decreased cost of substructure.
- Only four foundations are required for the entire span of 288 mts, which is impractical for conventional bridges.
- However structure like cable stayed bridge requires specialized equipments and highly skilled labour they are best option to cover large distances in deep channels where conventional bridges becomes impractical because of large number of substructures.
- Cable Stayed option for Railway over bridge in city limit can also be thought of as feasible option provided the height of pylon is not a problem.
- Major benefit of cable stayed bridge as ROB is the freedom of space available under the bridge for future alteration of railway lines.

FUTURE SCOPE OF WORK

- In the Present study steel plate girder type deck is considered, a similar study can be carried out taking various other options for deck like Prestressed slab type deck, steel truss type deck or concrete box type deck system.
- A study among different types of arrangement of cables like fan type and harp type can be carried out to know the variation of cost between these types of arrangement of bridge.
- A comparative study between cable stayed bridge and cable suspension bridge can be carried out.

- A study for investigating the range of cable stayed bridge as an substitute to other typical bridges can be taken.
- Wind tunnel experiment can be carried on the cable stayed bridge for fluttering and buffeting analysis of cable stayed bridge.
- A Complete dynamic analysis of the structure can be carried out to know the dynamic response of cable stayed bridge.

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