GUIDELINES FOR DESIGN, CONSTRUCTION AND MAINTENANCE OF CABLE STAYED BRIDGES

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NOTATIONS

Acronyms used are defined wherever these occur in the text and have been collected here for convenience.

V_{re}	Reduced Velocity
V	Velocity
B	A typical dimension, width of deck
S	Strouhal number
S_c	Scruton number
f	Frequency of Vibration, vortex shedding; subscript c,p pertain to cable, pylon respectively
D	A typical dimension for the cross section, subscript p pertains to pylon
d_c	Diameter of cable
F_L	Force due to lift
F_D	Force due to drag
F_M	Twisting moment
d	Depth of deck
L	Length of span
C_D	Coefficient of drag for a deck cross-section
C_L	Coefficient of lift for a deck cross-section
C_M	Coefficient of moment for a deck cross-section
α, α_w	Angle of incidence of wind
K_α	A torsional constant for the deck
f_t	First torsional mode frequency of bridge deck
f_v	First vertical mode frequency of bridge deck
I_u	Turbulence intensity
V_b	Basic Wind velocity
V_d	Design wind velocity
V_f	Critical wind velocity for classical flutter
V_{Rf}	Reduced critical wind speed
m	Mass per unit length of the element concerned

r	Polar radius of gyration of effective bridge deck cross-section at the centre of main-span
V_{crf}	Critical wind speed for initiation of single degree of freedom flutter
R_e	Reynolds number
h_a	The vertical vortex induced oscillatory amplitude
ζ_s	Critical damping ratio of bridge deck
β_{ds}	The shape correction coefficient
β_t	A coefficient, =1
Z	Reference altitude of deck, cable
z_0	Surface roughness
δ	Logarithmic decrement of damping
P_b	Aerodynamic susceptibility parameter
Sw	Mass damping parameter
k_1	Velocity conversion factor
V_H	Mean hourly wind velocity
G	Gust factor
ρ	Unit weight per unit length

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INTRODUCTION

Latest advancements and rapid developments in construction materials and machinery, design philosophy, construction technologies, have changed the concept about how road bridges are to be planned, designed, constructed and maintained. Bridge design philosophy is a set of assumptions and procedures which are used to meet the conditions of serviceability, safety, economy and functionality of the structure. Several design philosophies have been introduced from different parts of the world. The working stress method used earlier has since given way to the Limit State Method (LSM).

In view of the recent focus on accelerated construction of large number of special bridge structures like Suspension bridges, Cable Stayed Bridges, Extradosed Bridges, Long Span Arch Bridges & Long Span Cantilever Bridges, need was felt to prepare separate documents for such bridges. Accordingly, Steel and Composite Structures Committee (B-5) undertook the work of preparing Guidelines for Cable-supported Bridges and Structures (Part A: Cable Stayed including Extra-dosed Bridges") during its tenure 2015-2017 and prepared initial draft. While deliberating on the comments offered by the then B-9 Committee on the draft guidelines, the B-5 Committee in its 10th Meeting held on 22nd September, 2017 opined that there is a requirement of Code rather than a Guidelines and recommended that B-9 Committee takes over the responsibility of preparing the document for Cable Stayed Bridges alongside the document they devising for the Extradosed Bridges as this will reduce conflict and also cut down on the technological input to be made for preparation of these two highly needed documents for the engineering fraternity.

Thus preparation of a document for Cable Stayed Bridges was taken up to cover concepts, planning, aesthetics, design, analysis, wind and seismic effects, material, quality control, bearings, inspection, maintenance, structural health monitoring, etc. was undertaken by B-9 Committee during the tenure 2015-2017 headed by Shri G Sharan which carried on for two successive tenures 2018-2020 and 2021-2023 for devising such an exhaustive Guidelines alongside Guidelines for Extradosed Bridges. For this purpose, a Sub Group B-9.4 was constituted under the Chairmanship of Prof. Mahesh Tandon comprising experts viz Shri G Sharan, Prof. Prem Krishna, Dr. B.P. Bagish, Dr. N.C. Pal, Dr. B.C. Roy, Dr. Lakshmy Parmeswaran, Dr. Harshvardhan Subbarao, Prof. S.K. Thakkar, Prof. Yogendra Singh, Dr. Suresh Kumaresan Shri/s Inki Choi, V.N. Heggade, Vinay Gupta, Alok Bhowmick, C. Sankaralingam, Mohan Jatkar, U.K. Rajshirke, P.G. Venkatram, J.S. Pahuja, Nirav Mody and Jitendra Rathore. The Draft document on Cable Stayed Bridges was discussed in a number of meetings of the B9.4 Sub Group and B-9 Committee before it was finally deliberated and approved by the B-9 Committee in its meeting held on 19th September, 2022 for placing before the BSS Committee for its consideration, and, for forwarding it to IRC Council. The list of personnel of B-9 Committee for tenure 2021-23 is as under:

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The document finalized by the B-9 Committee was approved by the Bridges Specifications & Standards Committee (BSS) in its meeting held on 25th September, 2022 and subsequently, the Executive Committee approved the same in its meeting held on 7th October, 2022 and authorized Secretary General, IRC to place the same before the Council. The “Guidelines for Design, Construction and Maintenance of Cable Stayed Bridges” was approved by the IRC Council in its Annual Meeting held on 8th October, 2022 at Lucknow (U.P.) for printing.

The cable-stayed bridge form is eminently suited to spans in the range of 200 - 1000 m. The current record is held by the Russky Bridge in Vladivostok, Russia, with a main span of 1104 m., In Annexure A1, data of bridges in India, where available, has been summarised in a Table for ready reference. The cable stayed bridge form consists of a light slender deck supported by closely spaced stay cables which carry part of the load of the deck to the towers (pylons) that transfer the forces to the ground. The balance of the load is transferred to the pylons through flexure of the deck. The high structural efficiency of the form is derived from the light slender deck, the stay cables in tension, and the towers that transfer the cable forces to the foundations through vertical compression. The tensile forces in the stay cables also cause compression in the deck, often considered as ‘free prestressing’. Popularity of cable stayed bridges is also on account of the fact that enormous possibilities of aesthetics are offered by the form itself.

Slenderness of the deck has its complications in design because of the excitation of higher modes of vibration and the coupling thereof. While the light weight of the deck is of some advantage from seismic considerations, the opposite is the case for aerodynamic considerations. The main disadvantage of the cable stayed form is that the construction stage configuration is often more onerous than the service stage for transverse wind and it is not unusual to look for temporary stabilizing arrangements including dampers for long spans.

Chapter 1 of the Guidelines defines the scope in terms of maximum span length and the materials used for deck construction. The scope of these Guidelines shall be applicable for cable stayed bridges with maximum main span lengths as follows:

Concrete…….....upto 350 m

Composite and Steel……..upto 500 m

The Design Life of the Cable-Stayed Bridges will be taken as 100 years to be in sync with other current IRC codes.

Chapter 2 introduces the modern Cable-Stayed Bridge Construction with a brief history of its evolution. The typical features along with the flow of forces in the pylon, deck, and stay cables and the advantages and disadvantages of cable-stayed bridges are discussed.

Chapter 3 gives a brief overview of the structural forms of cable stayed bridge system and its three main components, i.e., the deck, the tower (pylon), and the inclined stay cables. The interaction of these components makes the structural behaviour of each cable- stayed bridge unique and fairly complex.

Chapter 4 describes the structural action of Cable-Stayed Bridges. This chapter explains various structural concepts through bridges that have been constructed in India.

The most suitable material for the decks of different main span ranges for Cable Stayed Bridges has been identified by Svenss on **[1]** as follows:

Concrete……...< 400 m

Composite…… 400 - 600 m

Steel…… .....> 600 m

Chapter 5 explains the construction methodologies adopted for cable stayed bridges. The cable-stayed bridges differ from conventional girder bridges, like use of cables, construction of upper pylon, installation and anchoring of cables, etc. The stresses and deformations produced in the members can be significantly influenced by the construction method and sequence of construction.

With regards to construction, it is necessary to satisfy required conditions such as environmental effects, construction safety, cost and period, which are based on its relation to entirety, purpose, type, scale and characteristics of project.

Prior construction planning is necessary for performance requirement of the project, in addition to the aspects of the quality and schedule control

As the influence of construction method, construction sequence and construction accuracy are significant in cable stayed bridges, chapter emphasizes on the importance of stationing the construction management team at site with necessary experience & sufficient knowledge.

Chapter 6 deals with Loads and Load Combinations to be considered for the cable stayed bridge. In general, the provisions of relevant codes for Concrete Bridges (IRC:112), Steel Bridges (IRC:21) and Steel- Concrete Composite Bridges (IRC:22) are applicable for structural design.

The weight of stay cables to be considered in the analysis and fatigue considerations has been specifically covered in this chapter since this does not find place in the above-mentioned codes. Similarly, the temperature difference between the stay cables and other parts of the bridge are given since the cables are more sensitive to temperature changes. The temperature difference between opposite sides of the pylon also must be considered in design for which guidelines have been specified.

The Accidental breakage of a stay cable must be considered as the sudden rupture of a highly stressed cable can result in dynamic augmentation of the force in the cable. The same is covered in this chapter.

The design life of the stay cable is much lower than that of the bridge as discussed in Chapter 10. This means that the stay cables would have to be replaced during the lifetime of the substructure, and this condition is to be considered for design. The allowable bending in the stay cable due to variable loads based on Fib 89 is covered here.

Although the Load Combinations for SLS and ULS checks must be carried out in accordance with IRC:6, the load factors and load combinations applicable for the Stay Cables have been tabulated separately in this chapter.

Chapter 7 of this guideline covers the analysis methods and design requirements for the design of cable stayed bridges.

Section-I, covers the structural modelling aspects for cable stayed bridges, including method to include the geometric nonlinearity of stay cables. Methods of analysis to obtain Second order effects in the pylon and deck which are slender elements are also covered in this chapter. The analysis method to be adopted for various limit states are explained. Finally, this section explains the methods to determine initial shape of bridge, obtain cable force (by preliminary calculations) and various construction stage analysis techniques which can be adopted during design stage of the cable stayed bridges.

Section-II of this chapter covers the design requirements of the stay cables for cable stayed bridges for Ultimate limit state, fatigue limit state and serviceability limit state. The fatigue consideration depending on the Wohlers Curve of stay cable is covered in this chapter. Based on Fib 89, the allowable stresses under SLS conditions shall be limited to 0.5 UTS, which shall form the basis of design. For ULS conditions, the strength shall be checked for UTS of the stay cable with a material factor of safety of 1.35.

In **Chapter 8** seismic effects have been examined. Cable stayed bridges are typically long span and flexible structures, requiring some special considerations for seismic design. Being

torsionally flexible and having coupled modes of vibration, a 3D modelling and analysis becomes necessary, considering adequate number of modes of vibration. In case, it is not possible to achieve 90% mass participation in modal analysis, a time history analysis may be required. These structures are designed to remain elastic (R=1) for the design basis earthquake (return period 500 years) with applicable importance factor. Having long spans and long lengths, cable stayed bridges are subjected to variation of ground motion underneath the different pylons/piers having coupled movement at the top due to continuity of the superstructure. These aspects have also been given an exposure.

In **Chapter 9** an effort has been made, to guide the reader in understanding the basics of Action and its Effects on Cable-Stayed Bridges. The various kinds of wind-induced oscillations (both limited and divergent amplitude) that can occur in cable-stayed bridge components, such as the deck, pylon, and cables, are briefly described. Also given is the methodology for checking the susceptibility of a cable-stayed bridge for wind-induced oscillations and to suggest the type of wind tunnel studies warranted.

Chapter 10 deals with Stay Cable Systems, Maintenance and Quality Assurance, covering in some detail, the corrosion & fretting protection requirements for the anchorages in transition zone and in the free length of the stay cables. Also included are levels of protection for preventing corrosion (internal barrier and external barrier) for various types of stay cable systems which are prevalent in newly constructed cable stayed bridges. Protection of stay cables from fire and vandalism are also emphasized. Various types of saddle systems used either for anchoring or passing of cable stay at pylon head with pros & cons for each saddle system have been included.

To mitigate vibrations in cable stays due to various effects as discussed in Chapter 10, different type of damping systems such as internal dampers, installed in connection with the cable anchorage and external dampers, installed independently of the anchorage and fixed to the deck are elaborated in this chapter.

Chapter 11 summarises the large variety of Bearings and Expansion Joints that can be employed in Cable Stayed Bridges. Given the length of continuous superstructure and the flexibility of the deck, movements at bearing and expansion joint can be as large as 1 m to 2 m. The bearings have often to be designed for rotations in the range of 0.05 – 0.15 radians. Hence, a highly durable and robust design and manufacturing of these devices becomes necessary and must be obtained from reputed manufacturers only. Apart from elastomeric, POT and Spherical bearings, Anti-Uplift Bearings are sometimes employed. If uplift is caused even under DL, it is usual to provide special devices which are mentioned in this Chapter. Since the design life of the bearings and expansion joints is much shorter than that of the bridge, arrangements for their replacement have to be incorporated. Modular expansion joints with full seismic movement capacity, seismic isolators, as well as devices like dampers of various types for different functionalities and STUs have also been discussed.

Chapter 12 deals with Geometry control which is an essential part of the construction monitoring process to ensure that finally the structure is translated in reality to the designed profile and alignment. The construction of cable stayed bridges, which involves stage construction of a flexible system, requires that the stresses, deformations in the structure and stay cable forces

meet design expectations during the construction as well as during service. The geometric control procedures for different types of bridges and kinds of construction methodologies differ. This chapter gives guidelines for casting curves during concrete casting, procedure during fabrication and trial assembly for the segments and also during erection of the deck.

Chapter 13 deals with Inspection, Maintenance and Structural Health Monitoring of cable stayed bridges. The various parameters to be monitored, identification of critical location, sensors, data acquisition system, data transmission and management have been discussed with case studies. For general bridge structure, the structural monitoring contents consist of four parts during the bridge operation, i.e. environmental monitoring around the bridge, loading monitoring, structural response monitoring under different loadings, and durability inspection monitoring.

Chapter 14 makes an attempt to highlight some of the features of cable stayed bridges that enhance its aesthetics to a level that cannot be surpassed by ordinary bridges. All the members are predominantly subjected to axial forces, with the pylon and the deck under compression and the cables under tension which leads to a very efficient structural system. The flow of forces in the structure are self-evident and that is part of the enchanting appearance of this concept of bridge construction. The additional elements of pylons and stay cables introduced in the structure have a huge visual impact. The size of the pylon with its enormous height becomes the most dominating feature of the bridge, and, most of the time even the landscape. There are Cable stayed bridges that have been built in the urban environment not necessarily because of requirement of a long span but to create a landmark structure. Lighting of the cable-stayed bridge at night always forms an integral part of the design.

CHAPTER - 1

SCOPE

1.1 The scope of this Guideline covers road cable stayed bridges of the following types:

- Steel Bridges
- Steel Concrete Composite Bridges
- Concrete bridges – Cast in place or Precast

These Guidelines include analysis, design, construction, testing, monitoring and maintenance aspects of cable stayed bridges. The foundations and abutments of these bridges shall be designed and constructed in accordance with well-established principles as per other relevant IRC publications.

This Guideline shall be applicable for cable stayed bridges with maximum main span length up to 350 m with concrete decks and 500 m with steel-concrete composite or steel decks, though certain aspects of longer spans are also addressed briefly.

This Guideline intends to help the designer to evolve concepts that would lead to

- A safe and technically sound design,
- A design that permits a faster construction time and reduced material consumption.
- A safe and economical erection method,
- Low maintenance with the required service life,
- An aesthetically pleasing form,
- A low impact on the environment

1.2 Base Document

Cable Bridges are the subject of several Guidelines such as FIB 89 [2], PTI [3], SETRA [4] and JRA (Japanese Specifications). These Guidelines have been formulated taking FIB 89 as the base document, but using the stipulations of Guidelines and Standards of the IRC and the BIS, to the extent available and applicable.

1.3 Design Life

The Design Life of the Cable Stayed Bridges will be taken as 100 years to be in sync with other IRC codes. However, in case it is desired to have a greater Design Life of the Cable Stayed Bridge, additional considerations come into play such as those enumerated below.

- Durability
- Discharge (and Scour)
- Seismic Effects
- Wind Effects

CHAPTER - 2
GENERAL

2 General Overview

The development of cable stayed bridges can be said to have begun with the construction of a bridge in Germany by Loscher in 1784. However, the principle of supporting a beam or a mast by taught cables goes far back in time. Early examples are bridges from natural materials such as bamboo for the beam and Lianas for ties. The masts of sailing vessels were always transversely stayed by shrouds. By the time of World War II, the era of modern cable-stayed bridge design was just about to commence. The first modern cable-stayed bridge was the Stromsund Bridge designed by F. Dischinger built in the 1950's in Sweden with a main span of 183 m and two symmetrical back spans of 75 m each. More than 1000 cable stayed bridges have now been built worldwide. A significant number of cable-stayed railway bridges have also been constructed. More recently very elegant pedestrian cable-stayed structures have been built and have become landmark structures. Today, cable-stayed bridges have firmly established their unrivalled position as efficient and cost - effective structural form in the 200 m to 1100 m span range and even more. The cost effectiveness and the general satisfaction with their aesthetic aspects have propelled this span range in either direction, with both increasingly shorter and increasingly longer spans being designed and constructed at the present time. Usual spans of CSBs are larger than those of extradosed bridges and shorter than suspension bridges for the reasons of structural design & economy, with overlaps either side. Cable-Stayed Bridges (CSB) have become increasingly popular in India over the last 40 years.

A cable stayed bridge consists of one or more tall columns (normally referred to as pylon or pylons, also called tower or towers), with cables supporting the bridge deck as shown in **Fig. 2.1 (d).** In the cable-stayed bridge, the pylons form the primary vertical load-bearing structure. In many cases there are no vertical load carrying bearings to support the deck at pylon location and the deck is suspended only through stay cables whilst in other cases vertical load carrying bearings are provided. However, the nature of support in the longitudinal and transverse directions at pylon and abutment and /or hold down piers can vary to suite design requirements and site constraints. The early engineering approach, up to World War II, to the stay cables was essentially derived and hybridized from the already established engineering experience with suspension cables and post-tensioning technology.

In a regular cable stayed bridge main span is the longer portion between successive pylons and the back spans (anchor spans) are the portion between pylon and abutment/hold down or anchor piers. Over a period of years, the arrangement of cables vis-à-vis their spacing along the span has undergone noticeable change. The Stromsund Bridge had only two cables on each side of the pylon, anchored to steel I-girders, which represents the trend in the 1950s. One of the reasons may have been the need to limit the degree of indeterminacy, and thus limit the computational effort required (modern computers had just begun to develop at that time). This required deeper girders to take up the bending moments generated. The modern trend is to have closely spaced (6 - 12 m) cables, thus

reducing the bending moments, depth of the girders as well as the size of the cables. This is one of the reasons for the range of cable stayed bridge spans having becoming longer.

Cable stayed bridges have evolved over time and one could say that we are in the fourth generation of cable stayed bridges today. The earlier CSBs of post world war 1950s are said to be the so called first generation. Over the years there have been considerable changes in the cable strengths i.e. the ultimate tensile strength, the development of parallel strands/wires technology, better understanding of the aerodynamic behavior, vast improvement in wind tunnel and CFD studies and a quantum leap in computing power, computer technology and analytical design software, improvements in construction and erection technology, and use of high strength concrete, Ultra High Performance Fiber Reinforced Concrete (UHPFRC), high strength steel – have all lead to the increase in span length over time, be it a concrete, steel composite or steel deck**. Fig. 2.1 (a), (b) & (c)** shows the evolution of span length over time for steel, concrete and steel composite cable stayed bridges.

Steel cable-stayed bridges	Record span length	Year completed
Rhine River Bridge Düsseldorf North	260 m	1957
Severins Bridge	301 m	1959
Knie Bridge	319 m	1969
Rhine River Bridge Duisburg-Neuenkamp	350 m	1970
St. Nazaire Bridge	404 m	1975
Normandy Bridge	856 m	1995
Tatara Bridge	890 m	1999
Stonecutters Bridge	1.018 m	2009
Sutong Bridge	1.088 m	2008
Russki Bridge	1.104 m	2012

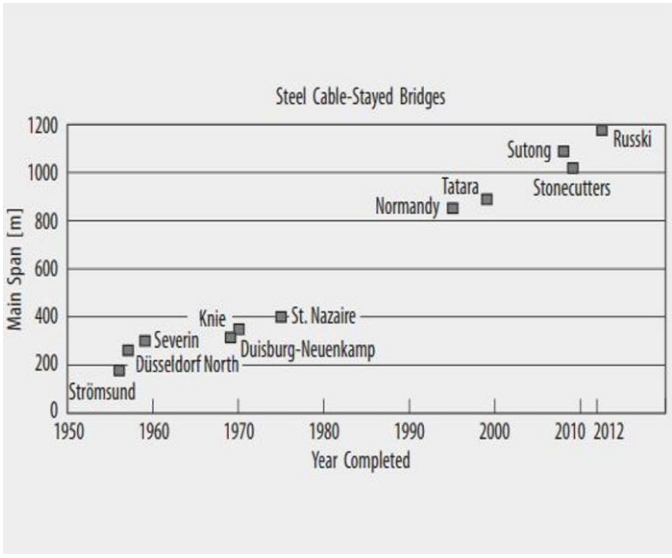


Fig. 2.1 (a) Record spans of Steel Cable Stayed Bridges

Concrete cable-stayed bridges	Main span	Completed
Maracaibo Bridge	235 m	1962
Wadi Kuf Bridge	282 m	1972
Brotonne Bridge	320 m	1977
Barrios de Luna Bridge	440 m	1983
Skarnsundet Bridge	530 m	1991

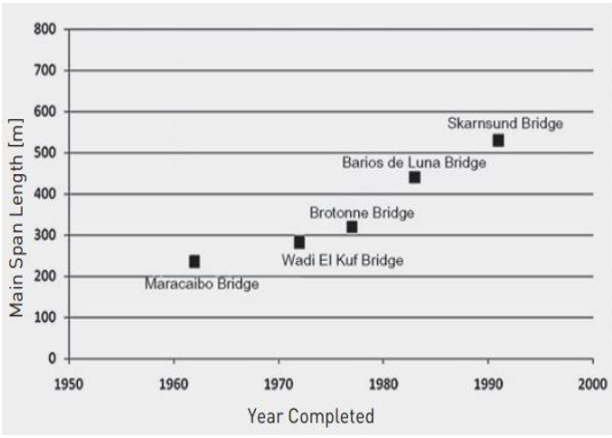


Fig. 2.1 (b) Record spans of Concrete Cable Stayed Bridges

Composite bridges	Main span	Completed
Hooghly River Bridge	457 m	1970 - 1994
Annacis Bridge	465 m	1986
Yang Pu Bridge	602 m	1993

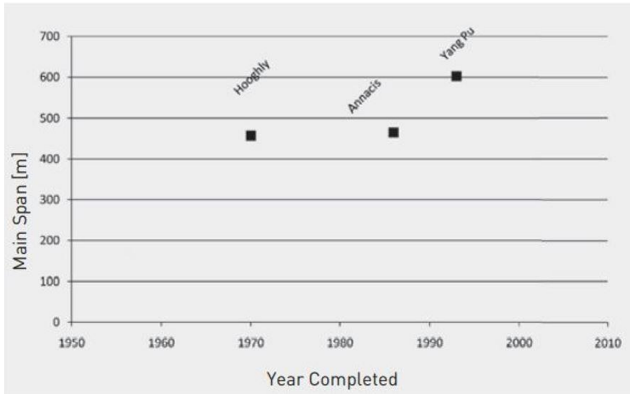


Fig. 2.1 (c) Record spans of Steel Composite Cable Stayed Bridges

The deck, as mentioned already, could be in reinforced or prestressed concrete, concrete-steel composite or in steel with an orthotropic deck. The choice would depend upon location, availability of material, labour and fabrication facilities.

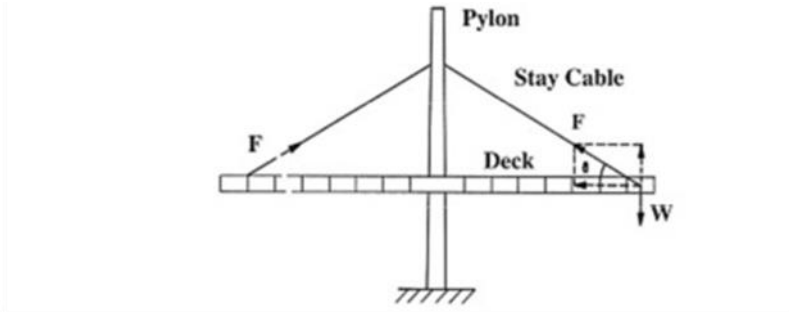


Fig. 2.1 (d) Triangle Structure: Deck-Pylon-Stay Cable [1]

The latest recommendations published by *fib* for stay cable systems **[2]** cover a significant number of new developments and specifications which forms the basis for this document and includes:

- Corrosion protection philosophy with a multi-barrier approach for the prestressing steel used for the stay cables and a rational design approach for structural steel components. A leak tightness test is also specified to verify the connection details between the free length and the anchorage zone of the stay cable and it is required at both ends of the stay and free length.

- Design and testing of stay cables for the inevitable flexural effects which occur close to the anchorages and other deviation points of the stay cables.

- Selected information on stay cable vibrations and special damping devices to control such vibrations.

- Suitable details for lightning protection of cable-stayed structures.

- Design consideration and testing procedures for stay cable saddles which are increasingly popular for cable-stayed structures with very slender pylons.

- State-of-the-art specifications for the main materials and components for stay cable systems including quality control procedures.

- Specific requirements for the common installation methods of stay cable are provided including the strand by strand installation and stressing methods.

- A comprehensive list of reference, relevant standards, and extended literature.

Post-Tensioning Institute, USA **[3]** and recommendations by the French Highway Administration, SETRA **[4]** have updated provisions on cable stayed bridges.

2.1 Features of CSB, Advantages and Limitations

The structure is made of three main components: Pylon, Deck, and Stay Cables. It can be compared to a triangular truss as in **Fig. 2.1 (d),** with the deck acting as the bottom chord. The stay cables are stressed at the time of erection in order to balance the weight of the deck. For subsequent loading conditions, the stay cables act as structural members as do the pylon and the deck.

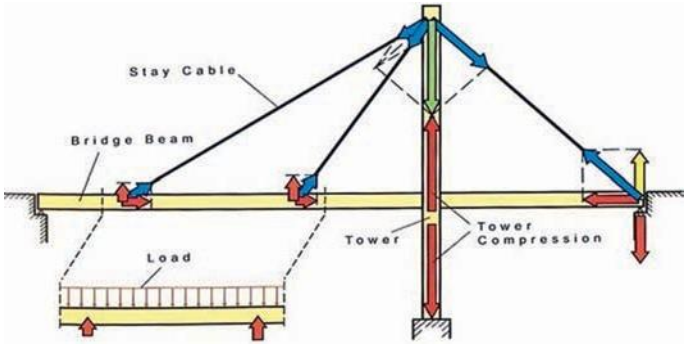


Fig. 2.2 Flow of Forces in CSB

Modern cable-stayed bridges have stay cables with well-defined and tuned cable forces which transfer their loads directly, **Fig. 2.2.** The horizontal components of the cable forces are introduced as compression forces into the deck. The forestay cables of the main span are tied back through the back stay cables to the ends of the bridge where these are anchored into hold-down (or anchor) piers. At the anchor point, each cable force is split into two components, the vertical force which is anchored to the ground and the horizontal force which is transmitted into the deck as a compressive force. The compression forces in the deck from the forestays and the backstays reach their maximum at the tower location.

CSBs offer a distinct advantage of being light yet stable, having shallow decks, aesthetically pleasing forms and are economical for spans of say, 200 m to 1100 m. However, while the central main span deck is suspended on one side of the pylon, the other side (the back span) cables are required to transmit the cable forces to the back span and balance the permanent loads to ensure overall structural stability. The longest back span cable is usually directly connected at the abutment and plays a vital role in the load balancing. Invariably, the back span is about 30% to 40% of the length of the main span in a regular CSB.

In case of Tatara Bridge, end span length was restricted due to existence of hills, hence, these were made in concrete, while the main span is in steel, in order to balance out the suspended load on both sides of the pylon. In case any of these features are not possible and the whole structure has to comprise of only one large span, then CSB is either not feasible or it requires special features like back stay anchored to ground (like Erasmus bridge in Holland, **Fig. 2.3**. Otherwise, an arch bridge or any other suitable form may have to be considered. Similarly, if a bridge/flyover with a large main span has to be located in the middle of a city, a CSB is not a preferred option from an aesthetical point of view as the pylon would cause a visual break in the urban habitat. However, these may be considered for ROBs where vertical clearance above rail tracks and matching the vertical profile to existing city roads are a constraint. In such cases, an Extradosed bridge may be more suited, instead of a CSB. In both cases, the depth of the deck is smaller than other conventional bridge types.



Fig. 2.3 Erasmus Bridge, Rotterdam, Holland

There are two main categories of modern concrete cable-stayed bridges:

1) **"Rigid" superstructure:** Box girders belong to this category. The box girder is used when high torsional rigidity is required: for instance, when the structure is supported with a central single plane of stay cables; **Fig. 2.4 (a).** Box girders may also be used to improve the aerodynamic stability of the bridge.

2) **"Flexible" superstructure:** This type of deck (with longitudinal edge girders as in **Fig. 2.4 (b)**, may be used when two or more planes of stay cables **Fig. 2.5** and **Fig. 2.6** support the superstructure and provide resistance to torsion. In Badhman CSB, three planes of cables have been used as in **Fig. 2.7**.

   The ratio of inertia between these two types of structures can be as much as 1/10.

   The flexible deck will more easily accept imposed deflections but will be more sensitive to buckling instability under high compression loads. The dead load of these two structural types is usually not much different. Longer steel box or

steel-composite decks are lighter and more flexible than similar cross section types in concrete decks.

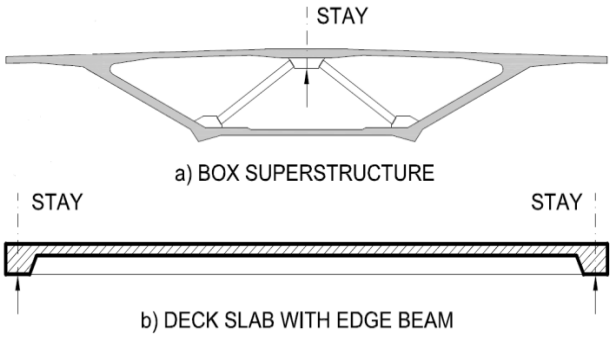


Fig. 2.4 Deck Girder

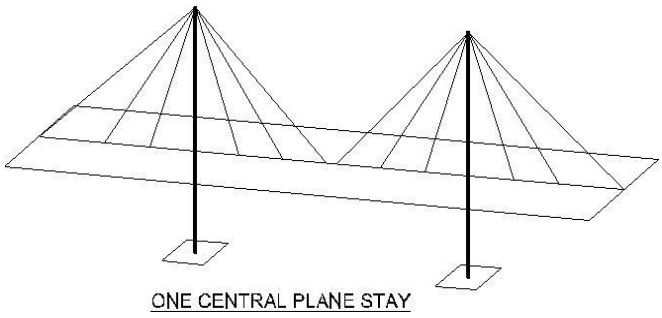


Fig. 2.5 Single Central Plane of Cables

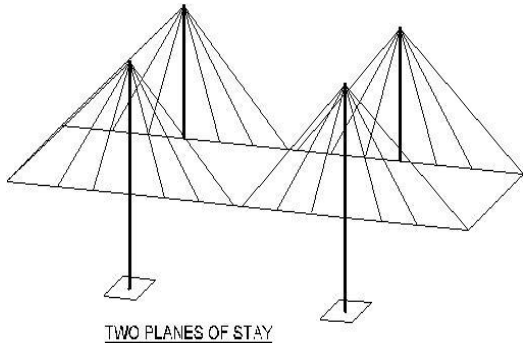


Fig. 2.6 Two Planes of Stay Cables

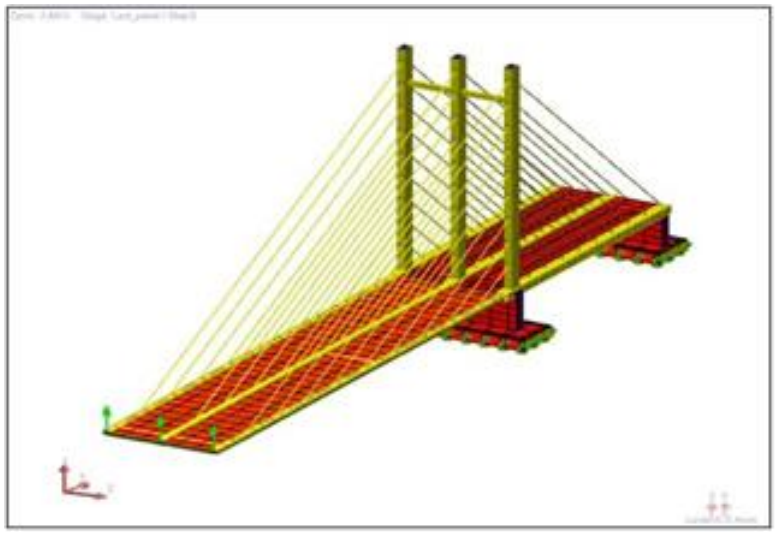


Fig. 2.7 Multi-plane Cables

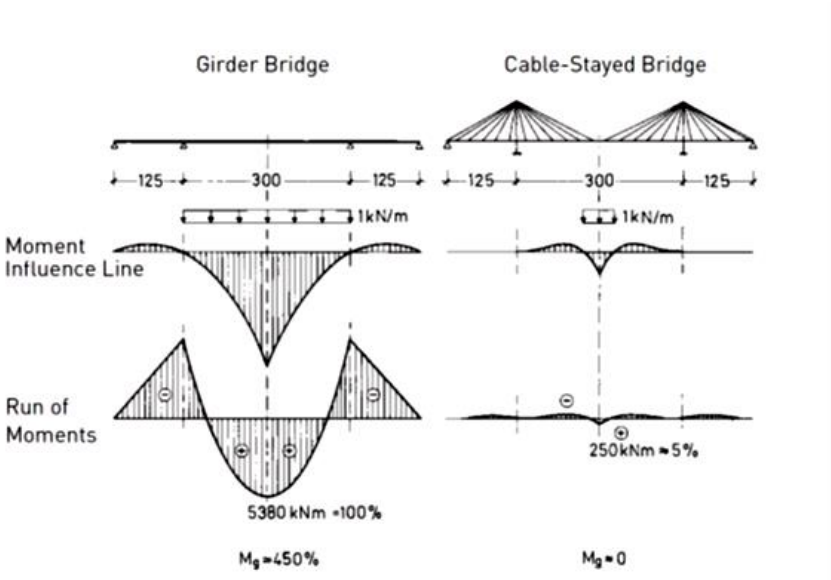


Fig. 2.8 Comparison of Beam Moments [1]

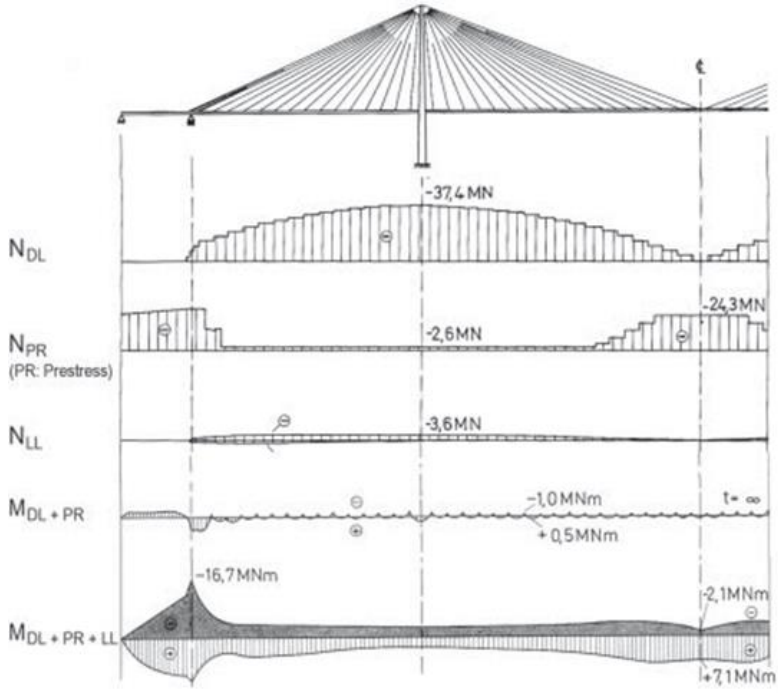


Fig. 2.9 Bending Moments and Compression Forces in Beams of Cable Stayed Bridges [1]

2.2 Advantages of CSB

- The major advantage of CSB is that the stay cables which are a part of the completed structure also serve the function during the construction stages. The flow of forces in the stay cables remains by and large the same.

- Another main advantage of CSB is that the bending moments are greatly reduced by the load transfer of the stay cables similar to a beam on elastic foundation, **Fig. 2.8** By installing the stay cables with their predetermined precise lengths, the support conditions for a beam elastically supported (idealised as non-yielding support) at the cable anchor points can be achieved and thus the moments from permanent loads are minimized, **Fig. 2.9.**

- Large compression forces in the beam are caused by the horizontal components of the inclined stay cables. The normal forces in the main and side span equal one another so that only uplift forces have to be anchored in the abutments which act as hold-down piers.

- Negative live load moments may occur over the vertical bearings at the towers. These can be avoided by supporting the beam by the stay cables only, even in the tower region. The biggest positive and negative moments occur in the side spans near the hold-down piers, which may require special measures. Shear forces remain small.

- The largest horizontal components belong to the backstay cables, **Fig. 2.9** The beam compression forces thus increase quickly at the beam ends and increase less quickly near the towers where they reach a maximum. For concrete decks this compression force is beneficial. Only in the center of a 3-span configuration, where the compression forces from cables are small, longitudinal post-tensioning may be required, which also has to overcome tension from friction, braking, etc.

2.3 Comparison of CSB with Suspension Bridge and Extradosed Bridge

A Suspension bridge has a system where there are two sets of main suspension cables, acting as a catenary, connected to pylons on either side of the span. Pylon heights are shorter than for cable stayed bridges, as the pylon of a suspension bridge has to mainly cater to sag of the catenary. This catenary suspends many auxiliary vertical cables to suspend the deck at close intervals. This type of system is suitable for larger spans of say 500 m to 2000 m.

Fig. 2.10 draws a comparison between steel deck cable-stayed and suspension bridge quantities of similar span lengths for a preliminary estimation based on a theoretical exercise. At 1000 m main span, the quantities of steel suspension bridge are more than twice that in a CSB, which demonstrates the efficiency of CSB.

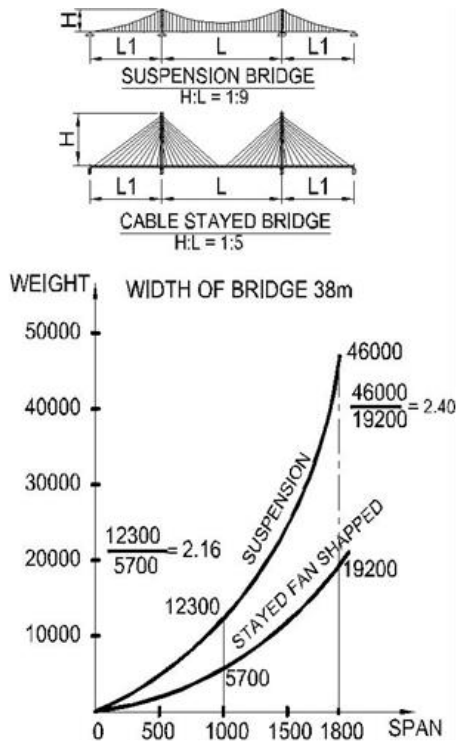


Fig. 2.10 Quantity vs Span Length for Steel / Steel Composite Cable Stayed and Suspension Bridges [5]

Although, in theory the design of span length 1400 m or more is possible with cable stay bridges, but in practice, for span ranges 1200 m-1400 m the choice between suspension and cable-stay will depend upon a number of factors such as topography, construction aspects, economy, etc. Recently, hybrid of suspension – cable stay bridge system has been revived for very long spans, for example Sultan Suleiman bridge (1500 m main span) in Turkey, Messina Straits Crossing (3300 m main span) in Italy, which are on the anvil.

a) Suspension bridge

b) Cable stayed bridge with twin towers

c) Cable stayed bridge with A- tower

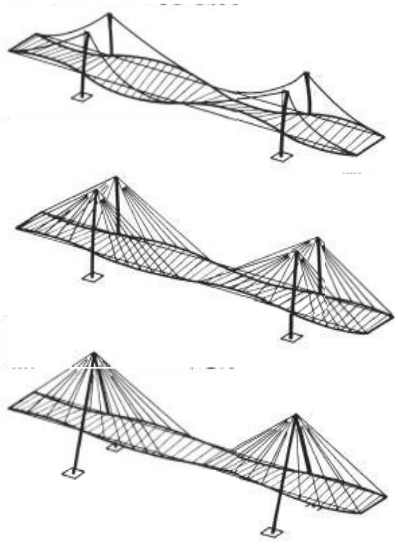


Fig. 2.11 Eigen modes of Suspension Bridges and Cable-Stayed Bridges [5]

Cable-stayed bridges are inherently stiffer than suspension bridges. This is especially true for longitudinally eccentric loads, for which the main cables of a suspension bridge find a new equilibrium without increase of cable stresses (stressless deformations), whereas stay cables always have to be additionally stressed in order to carry any load.

Suspension bridges are thus generally not suitable for high railway loads due to which deformations become too large.

The eigen frequencies of cable-stayed bridges are significantly higher than those of suspension bridges. The torsional frequencies, which are especially important for the aerodynamic safety against flutter, can further be increased by using A-towers which omit the elasticity of the backstay cables, **Fig. 2.11**.

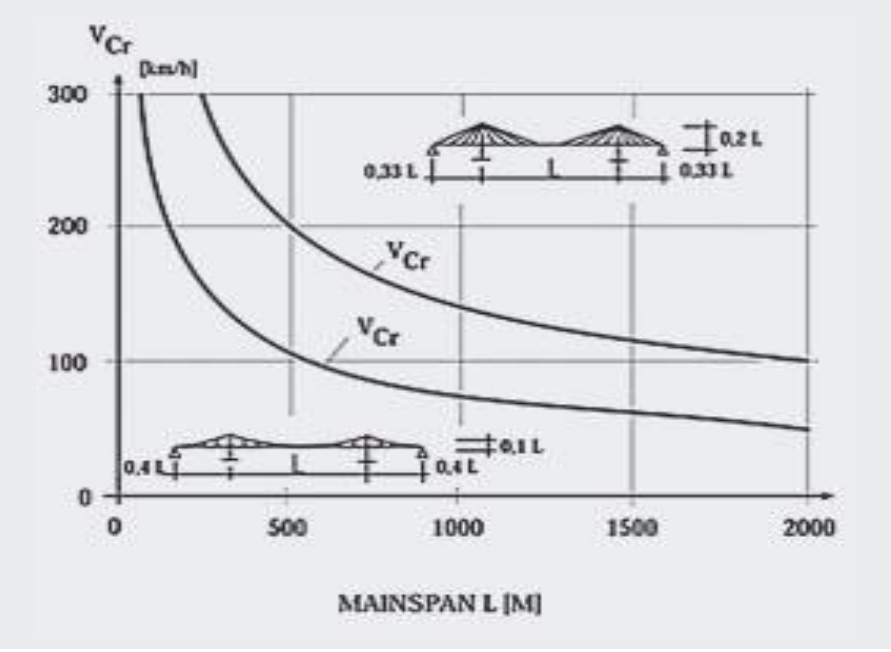


Fig. 2.12 Critical wind Speeds of Suspension Steel / Steel Composite Bridges and Cable-Stayed Bridge [1]

The critical wind speed for the onset of flutter instability due to wind induced vibrations is thus higher for cable-stayed bridges than for suspension bridges. The diagram in **Fig. 2.12** shows that for a main span of 500 m the critical flutter wind speed for a typical cable-stayed bridge is about 200 km/h while for a corresponding suspension bridge it is only 100 km/h.

Cable stayed bridges are quite different from extradosed bridges. Basically, extradosed bridges are in between girder bridges and cable stayed bridges. The extradosed cables are in fact externally prestressed cables. The cables and anchorages used for extradosed bridges are usually different compared to cable stayed bridges, for the reasons of economy.

Extradosed bridges are suited to smaller span range, say 50 m to 250 m. Similar to cable stayed bridges, these require end spans of about half of central main span. EDs are more rigid and less susceptible to cable vibrations and cable fatigue compared to CSBs. Required height of pylon is the least among the three forms of cable supported bridges. Extradosed bridges are comparatively economical for span ranges of 100 m to 200 m.

Annexure A1 of this guideline gives a table containing broad details of most of the cable stayed bridges in India so far which gives an idea that India already has cable stayed bridges for 40 years.

CHAPTER - 3
STRUCTURAL FORM

3.1 Structural Scheme & Span Configuration

The economic main span range for Cable-Stayed Bridges lies between 200 m and 1100 m. The arrangement of stay cables is influenced by the relationship between the main span and the side span and the location of backstay anchorage. Usually, a module for CSB will comprise of 3 span continuous structure with two towers. However, there are exceptions and cable-stayed bridges can also have two spans with single tower as well as multiple spans.

For conventional 3-span modules, the ratio between the side span and main span has an important influence on the stress changes in the backstay cables which hold the tower back to the anchor pier. Live load on the main span increases these stresses whereas live load on the side span decreases them. These stress changes must be kept safely below the fatigue strength of the cables. From consideration of fatigue strength of stay cables and live load/dead load ratio, the side span is generally kept about 40% in length of the Main span.

Modern cable-stayed bridges have used both steel and concrete material in deck and pylons. Configuration for towers include single- and double-plane vertical, A-shaped, and double-plane sloping towers. Deck schemes include cast-in-place concrete, precast concrete, orthotropic steel decks, composite decks, and prefabricated steel. Cable configurations include radiating, harp, and fan. Cable systems use high-tensile bars, parallel wires, or steel bars protected by polyethylene or steel pipe.

Finalisation of the structural scheme and span configuration of a cable stayed bridge is carried out at the planning and conceptual design stage of the project. The concept design evolved must meet the owner's established requirements of aesthetics, operability, economics, maintainability, and functionality. Bridge design requires involvement of experienced personnel knowledgeable in state-of-the-art techniques and methodologies. Important considerations for a cable-stayed bridge design include:

a) A concept which is safe and technically sound,

b) A construction and erection method, which is safe, practical, construction friendly.

c) A structure with low maintenance needs and with a long service life,

d) A structure with aesthetically pleasing form, with low impact on the environment, and

e) A design that permits a faster construction time compared to an alternative comprising the same material and manpower input.

Cable-stayed bridges may be classified by the type of material they are constructed of, by the number of spans stay-supported, by transverse arrangement of cable-stay planes, and by the longitudinal stay geometry.

There are two main categories of modern concrete cable-stayed bridges, namely Rigid Superstructure and Flexible Superstructure, which is explained in detail in **Chapter 3** and shown in **Fig. 3.1** below.

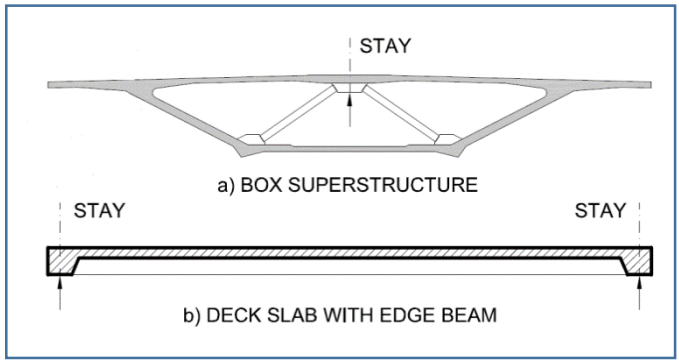


Fig. 3.1 Categories of Modern Cable Stayed Bridges

3.2 Tower Shapes

- A-shaped Pylon
- Diamond Shaped Pylon
- Needle Pylon
- Goal Post Pylon
- H-Shaped Pylon

Fig. 3.2 shows various shapes

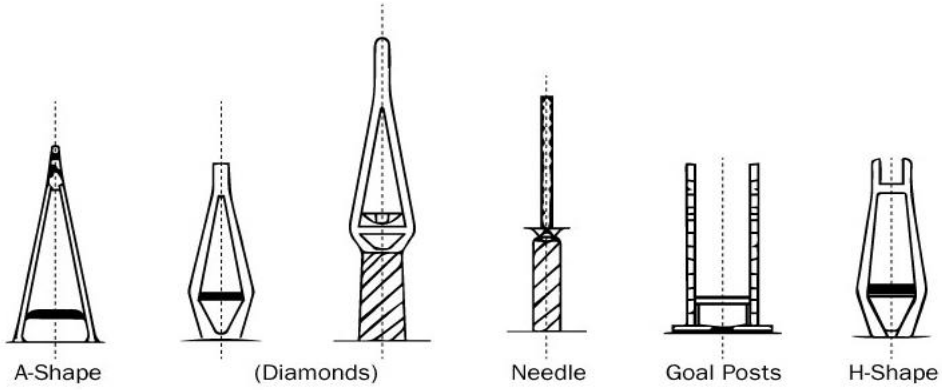


Fig. 3.2 Categories of Modern Cable Stayed Bridges

Towers constructed from concrete are normally the most economical system. Steel towers are used only in rare cases where there are special considerations.

In the longitudinal direction, the tower should be slender and have a small bending stiffness to avoid large bending moments and reactions at the foundations. Tower bracing above the road deck should be slender and appear light between the thin cables.

Inclined tower construction requires temporary ties to prevent collapse. The towers may need to be cast with an outward chamber for dead load deflection caused by the inclination.

Towers providing live or dead end cable anchorages should be readily accessible for cable installation, stressing, and grouting. This includes providing sufficient space for personnel and equipment necessary to complete these activities. If tower prestressing exists, the same access criteria is required.

Loads induced on the tower by temporary crane tie-ins need to be checked so that allowable stresses are not exceeded.

3.3 Deck Material

The primary construction materials used in the deck of cable-stayed bridges are reinforced or prestressed concrete, composite concrete-steel, or orthotropic steel decks. The choice of deck material will depend on the overall concept for the structure and its elements: the selection of the appropriate structural system for the bridge considering its specific function, site location, and required spans. A well-selected concept determines the efficiency and economy of the bridge, saves materials, cost, and construction time. Good design concepts minimize problems and future difficulties both in the design office and on the construction site.

3.4 Deck Cross Sections

Deck cross-sections are classified by the type of material used in the construction: concrete, steel, and composite steel-concrete. In addition, deck cross-sections may also be classified as a hybrid deck when the main and side spans have different materials. In this case, the main span is usually made of steel in order to be lighter in weight, and the side spans are in concrete and work as a counterweight to provide stability for the main span. **Fig. 3.3** to **Fig. 3.5** show some deck cross-section geometries that are used depending on the type of material.

Besides the geometry, the length of the cable-stayed bridge main span has a direct relation to the type of material. Essentially, concrete decks are for short spans (upto say 400 m), steel decks are for long spans (for spans beyond 700 m) and composite steel-concrete are used for intermediate spans. The record of longest main span of cable-stayed bridges **Table 3.1** is 530 m, 616 m and 1104 m for concrete, composite and steel deck cross-sections, respectively.

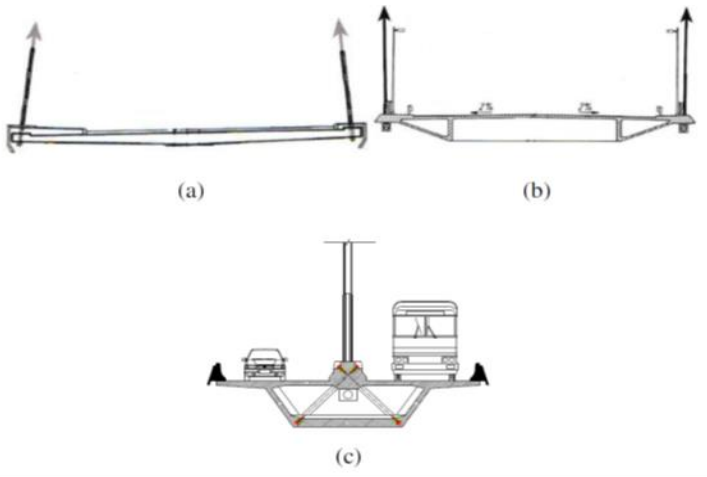


Fig. 3.3 Concrete cross-sections: (a) Thin Concrete Beams (Koppel, 1984, Svensson, 2012); (b) 2 Concrete Girders (Leonhardt, 1980, Svensson, 2012); (c) Box Girder (Battista, 2011)

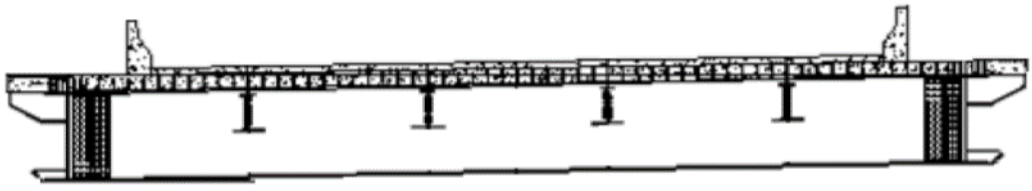


Fig. 3.4 Composite Steel-concrete Cross-Section: Two Main Plate Girders (Battista, 2013)



Fig. 3.5 Steel cross-section: box girder (You et al., 2008, Svensson, 2012);

Table 3.1 Record Main Spans for Cable-Stayed Bridges

Material	Bridge	Country	Year	Main Span (m)
Concrete	Skarnsundet Bridge	Norway	1991	530
Composite	Erqi Yangtze River Bridge	China	2011	616
Steel	Russky Island Bridge	Russia	2012	1104

Table 3.2 shows the relation between type of cross-section and main span size. To develop this relation, it was assumed that short and medium main span lengths are under 200 m and 450 m, respectively, while long spans are over 450 m.

Table 3.2 Relation Between Main Span Length and Cross-Section Material and Geometry

Material	Geometry	Main Span Length		
		Short	Medium	Long
Concrete	Thin Concrete Beams	√		
	2 Concrete Girders	√	√	
	Box Girder		√	
Composite	2 Plate Girders	√	√	√
	3 Plate Girders		√	
	4 Plate Girders		√	
	Truss Deck		√	
	2 Box Girders			√
	Hybrid			√
Steel	Box Girder		√	√
	2 Box Girders		√	√

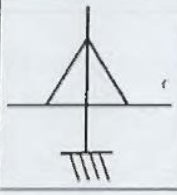
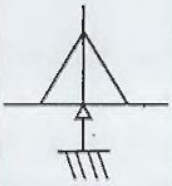
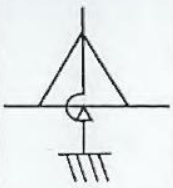
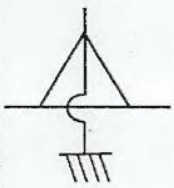
In selecting the cross section of the deck for cable stayed bridges, in addition to the bridge type and load, the number of stay cable planes, the way they will be anchored, and the transfer of tension between the cables and the girder shall all be considered. Also, the deck cross section shall be determined with consideration for wind resistant stability.

3.5 Connection between Pylon, Pier and Deck

- Pylon, Pier and Deck are all rigidly connected
- Pylon and Pier are rigidly linked. Deck is free or support bearing is put under the deck
- Pylon and girder are rigidly linked, and support bearing is put on pier top.

Table 3.3 gives the comparison between various types of girder support and the connection between Pylon, Deck and Pier **[2].**

Table 3.3 Comparison of Girder Support Types and Connection Methods for Pylon, Pier & Girder

Type of girder support	Connection of pylon, pier and girder	Outline Diagram	Characteristics
Rigid frame type	Pylon, pier, and girder are all rigidly connected		• Bearing is unnecessary, suitable for cantilever construction method. • If more than 3 spans, it takes large effects by temperature change, creep, and shrinkage. • Little displacement during earthquakes but sectional force of the girder in the region near the pier top is large.
Continuous girder type	Pylon and girder are rigidly connected; Pylon and girder are supported by bearing		• Reaction force distribution for piers becomes easier. • Cross section of girder and pylon in the region of connection section becomes larger • The bearing which supports the pylon and girder need a large scale. • During cantilever construction temporary fixing is necessary. • Sectional force of superstructure is small during earthquakes. • The first mode is dominant during earthquakes and natural period becomes shorter. • Acting position to the substructure becomes low
	Pylon and pier are rigidly connected; girder is supported by bearing		• It is possible to distribute the reaction force to piers • The bearing which supports the part of girder can be done by small scale. • Suitable for continuous type girder structures • During cantilever construction temporary fixing is necessary. • Sectional force for pylon and pier becomes large during earthquakes.
Floating type	Pylon and pier are rigidly connected; girder is not supported by bearing		• Long natural period, can lighten earthquake forces but large girder displacement. • Bearing support is not necessary but the bearing fixing the girder in direction perpendicular to bridge axis is separately necessary. • During cantilever construction temporary fixing is necessary. • Limited to multi-cable cable-stayed bridges. • Sectional force for pylon and pier becomes large during earthquakes.

3.6 Stay Cable System and Technology

- Number of Cable Plane
  - Single Plane
  - Double Plane
- Arrangement of Cables
  - Harp Shape
  - Fan Shape
  - Radial Shape (Parabolic Shape)

There are three basic cable arrangements in bridge elevation. With the fan arrangement, all cables join at the tower head **Fig. 3.6.** With the harp arrangement, all cables run parallel and are anchored over the height of the tower **Fig. 3.7.**



Fig. 3.6

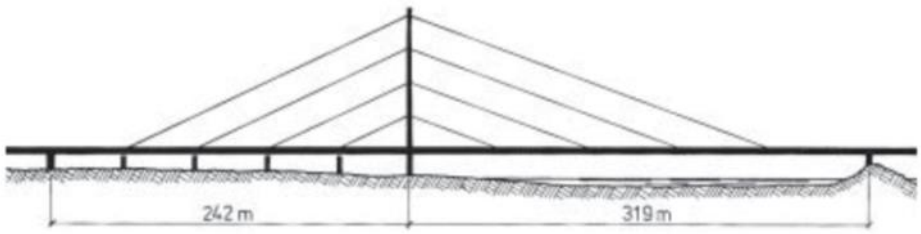


Fig. 3.7

The true fan arrangement cannot be realized practically since in the case of damage, each individual cable must be able to be exchanged by reversing the installation procedure, which is difficult in true fan arrangement. This exchangeability requires individual cable anchorages to have certain minimum spacing between them at the tower head.

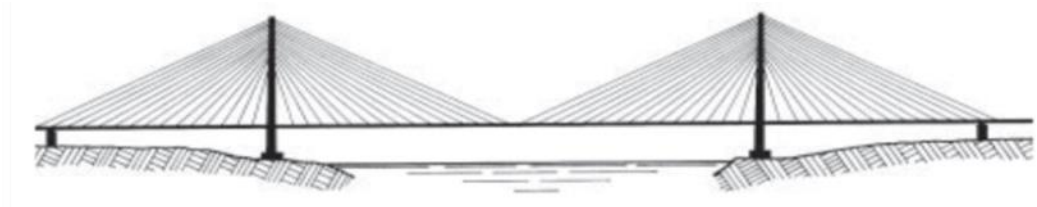


Fig. 3.8

An intermediate cable arrangement between harp and fan is created, as shown in **Fig. 3.8**, which should be as close as possible to the fan arrangement due to its economical advantages. The harp arrangement, however, is aesthetically advantageous because there are no visual intersections of the stay cables, even for two cable planes in a skew view.

The arrangement of stay cables is also influenced by the relationship between side span and main span, and the location of the backstay anchorages, e. g. by connecting them directly above the piers in the side spans. The harmonic cable arrangement plays an important role in the aesthetic appearance of a cable-stayed bridge and has thus to be studied carefully from the very beginning of a design.

CHAPTER - 4
CONCEPTUAL DESIGN

4.1 The Concept

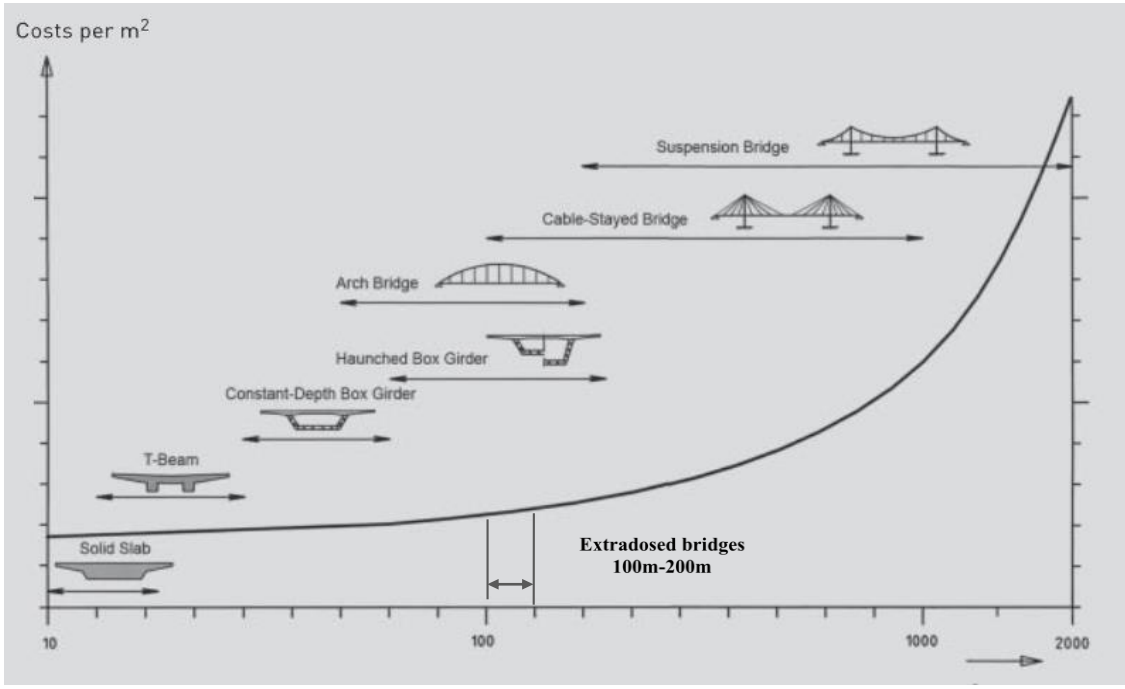


Fig. 4.1 Economical Span Range for Various Bridge Types

Each bridge type has a range of spans in which it is most economical. This is shown in **Fig. 4.1**, modified from Svensson **[1].** Svensson has also come out with the most suitable material for the decks of different span ranges for Cable Stayed Bridges as follows:

Concrete……. < 400 m

Composite…… 400-600 m

Steel…… > 600 m

Good references for the Preliminary design of Cable Stayed Bridges are already available in several books and papers Svensson, Gimsing, Walther, Troitsky, Calvi **[1-5]**, to name a few. The attempt in this Chapter is to understand the structural concept of Cable Stayed Bridges by viewing it largely through the bridges that have been constructed in India.

The superstructure of Cable stayed Bridges **Fig.4.2** consists of the following:

- Deck
- Pylons
- Stay Cables

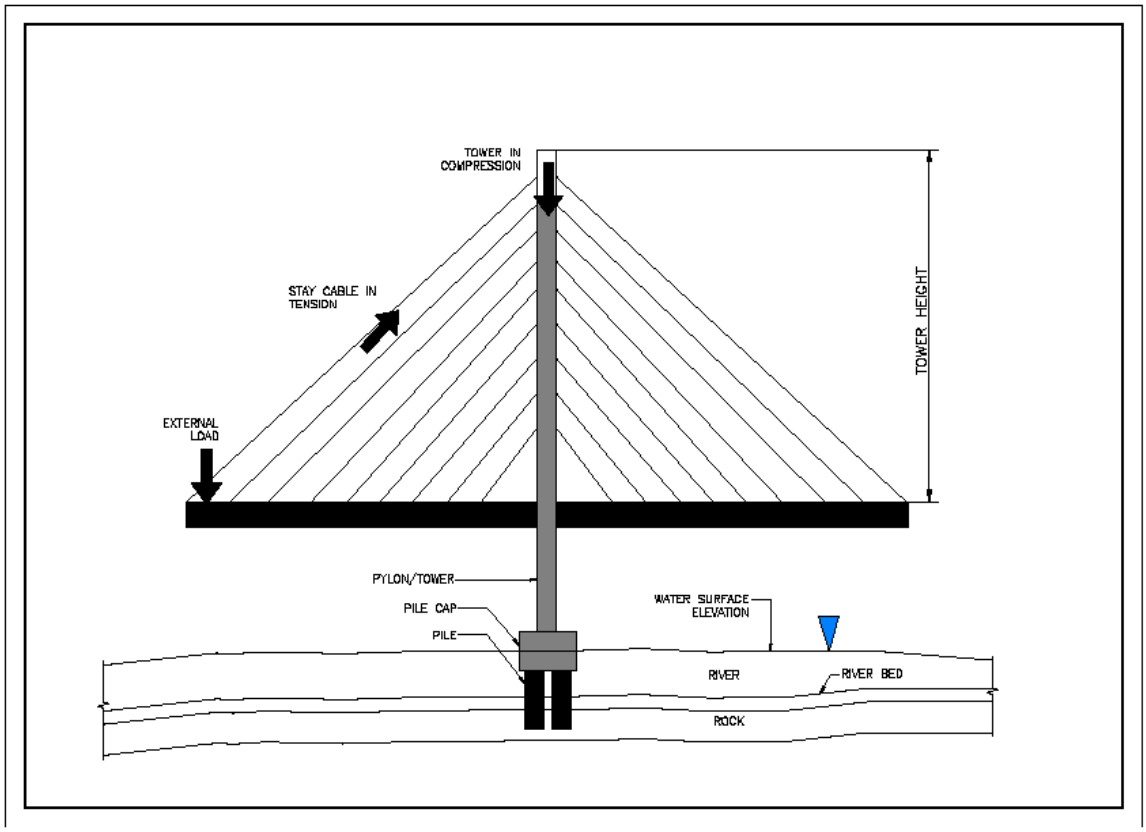


Fig. 4.2 Structural Action of Cable Stayed Bridges

The structural actions of all three are linked when we take account of dead loads of the structure or when a vehicle crosses over the deck. The global structural action can be seen in **Fig. 4.2** clearly. Any load on the deck (including its own dead weight) is carried mostly by the tension in the stay cables and only a small part of it is carried through the flexure of the girder. The vertical loads are transferred to the pylon which is designed to carry the axial compression imposed on it. The Compressive axial forces in the deck caused by the inclined stay cables and the axial compression in the pylons are in equilibrium with the tensile forces in the stay cables. Since the loads are mainly transferred as axial forces rather than bending, it makes the option of Cable Stayed Construction most economical for long span bridges. It is also simpler and safer to construct as the deck can be constructed using the free cantilevering technique without ground support. It is faster to construct because the erection of the deck can be commenced as soon as the pylons reach the height of the first cable anchor point, while the remaining height of the pylon and the deck erection can be carried on simultaneously.

It is highlighted that bearings are almost always preferred to be avoided at tower locations so that hogging moments due to vertical live load do not occur and the deck is ‘cradled’ by the stay cables by providing elastic supports to the deck. Bearings are provided at the end supports i.e. abutments in a three-span arrangement, **Fig. 4.3** for vertical loads apart from providing restraints at all supports for lateral movements transverse to the deck. The deck can move longitudinally under the effect of temperature changes, shrinkage, creep or live loads and hence one of the end bearings has to be made a “rocker” while at the other end a “free” bearing is provided. Given the long length of the bridge, expansion joints capable of large movements become necessary.

The three-span arrangement (**Chapter 3**) is the most common arrangement employed in Cable Stayed Bridges which consists of a main span and two side spans. A comparison of the three span arrangement for the common bridge types **[6]** is shown in **Fig. 4.3**. It may be noted that the side spans of Cable Stayed Bridges used for highways are optimized as being about 40% of the main span in contrast to the Free Cantilever and Extradosed Bridges where they are much longer. The reason is that the sagging effects of the main span in Cable Stayed Bridges often become the governing criteria of design and a shorter side span would provide greater stiffness to the main span. A comparison of midspan effects of the main span for a simply supported beam and a fixed ended beam is shown in **Fig. 4.4** to illustrate the importance of providing greater stiffness at the ends of the main span. As shown, the deflection in the main span in the fixed beam is merely 1/5th of that of the simply supported case, while the bending moments reduce to 1/3rd. However, the short side spans result in a tendency of the deck to uplift at the end supports both due to DL alone as well as LL when applied on the main span. This necessitates the anchoring of the deck at the end support by kentledge or other means which will be discussed later in this chapter. The variation of stress in the backstay cable is another issue which will also be discussed.

In some instances, short multiple spans are provided in the approaches instead of one single side span which improves the situation still further **Figs. 4.5 & 4.6** (Chambal Bridge). There is an additional advantage of this arrangement in that the stay cables can be made symmetrical about the pylon which avoids large stress variation in the backstays. This is discussed later in the Chapter.

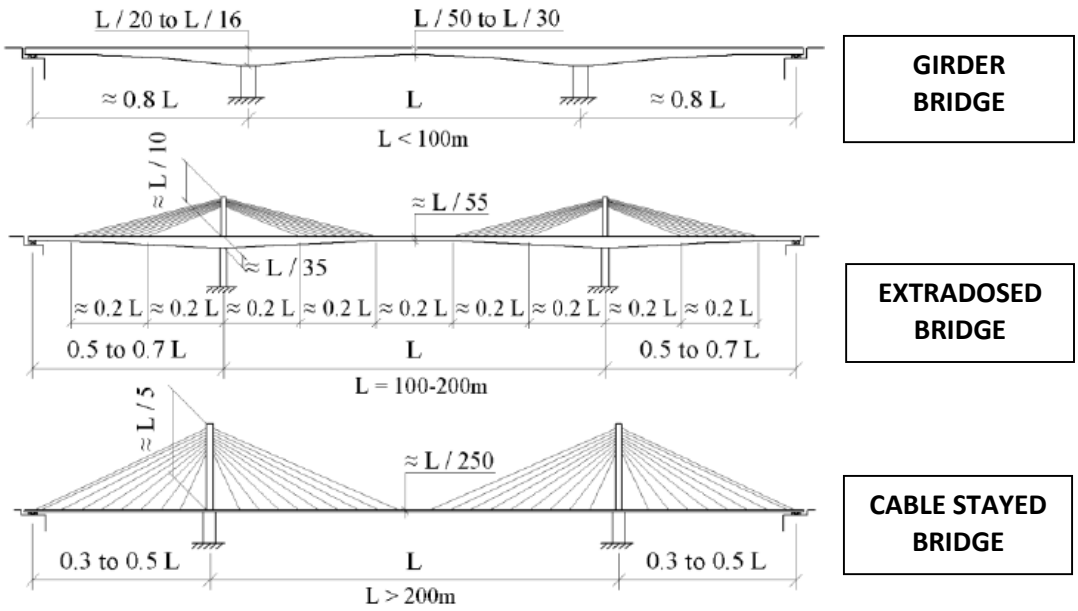
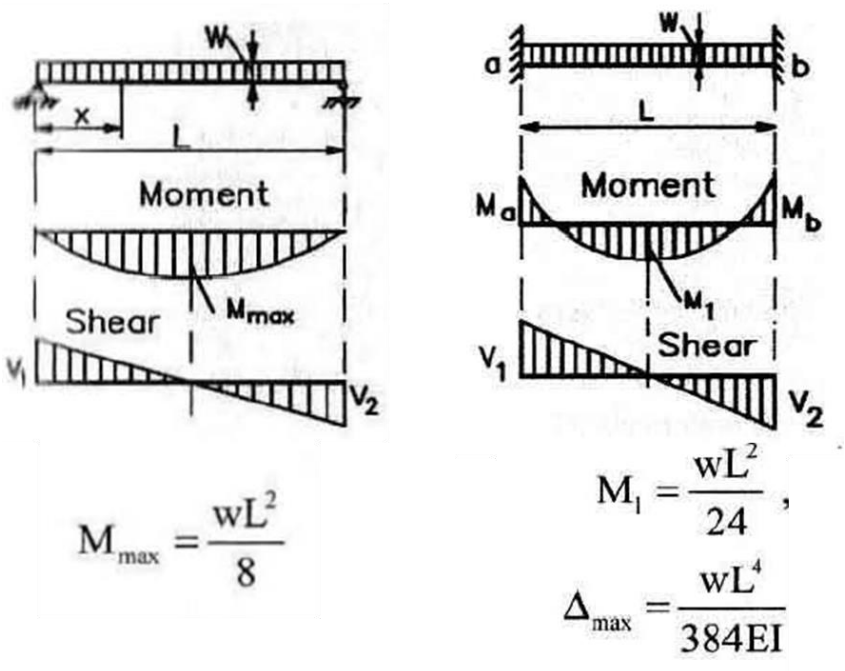


Fig. 4.3 Proportioning of Spans in Different Bridge Types Compared [6]



$$M_{max} = \frac{wL^2}{8}$$

$$M_1 = \frac{wL^2}{24},$$

$$\Delta_{max} = \frac{wL^4}{384EI}$$

Fig. 4.4. Importance of Stiffness Provided by Side Spans for Reducing Sagging Effects in Main Spans

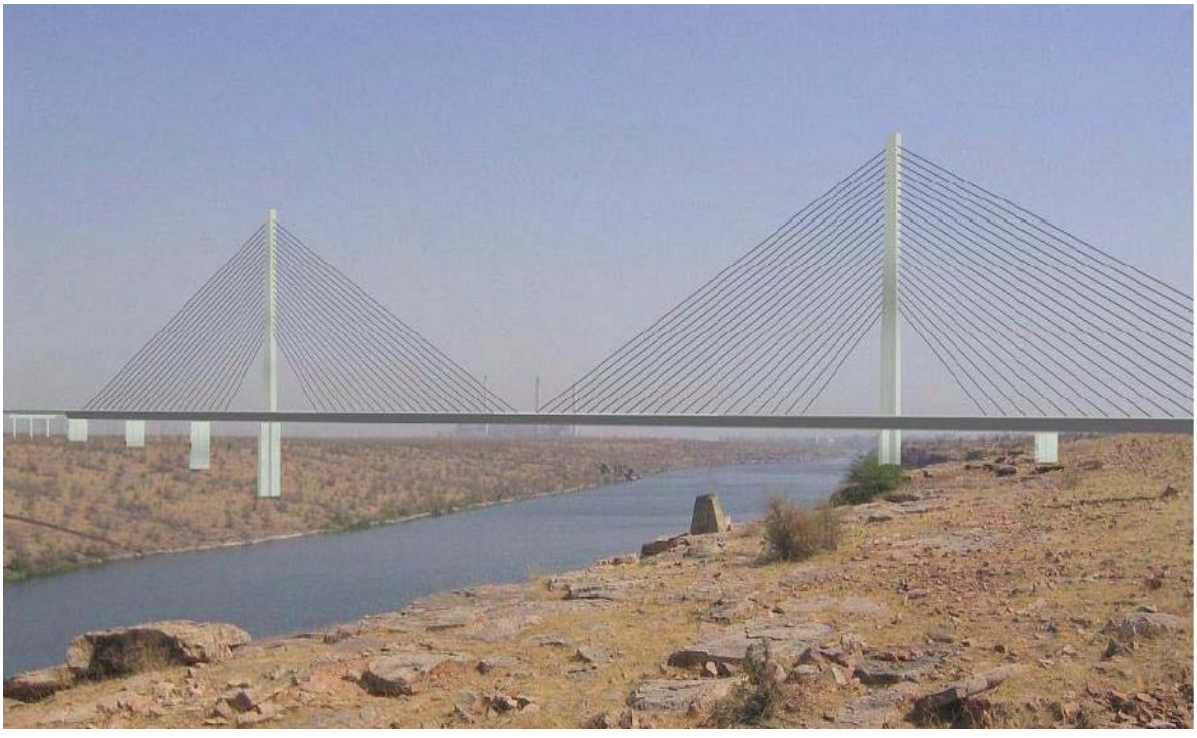


Fig. 4.5: Kota Bridge over Chambal, Rajasthan (350 m)

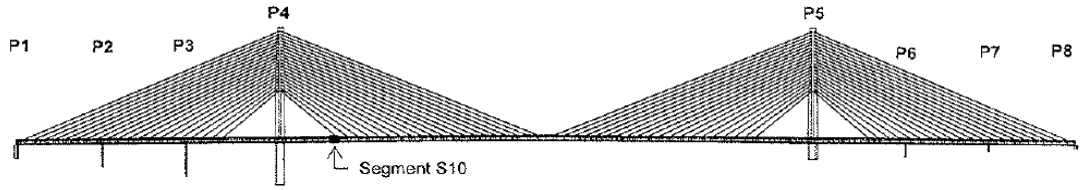


Fig. 4.6. Kota Bridge over Chambal, Rajasthan (Main Span: 350 m, Side Spans 50 - 60 m)

In the case of multi-span cable stayed bridges with equal spans the benefit of short side spans cannot be availed of. The same is the case with Single Pylon bridges supporting two equal spans on either side. Since there are no backstays in such structures which can restrain the tower head, the tower has to be stiff by itself. This is best achieved by towers that are A-shaped in the longitudinal direction, **Fig. 4.7 (a) + (b)** (Worli Bandra Bridge).

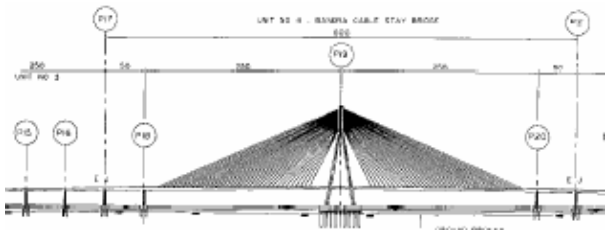
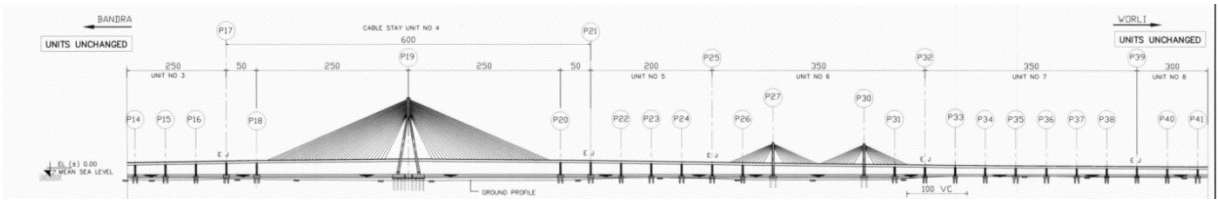


Fig. 4.7 (a) Worli Bandra Sea Link





2*250 m span with
Single pylon

Length: 5.6 km
Width : 8 lane traffic

Fig. 4.7 (b) Worli Bandra Sea Link, Mumbai

The design of such bridges is discussed later in this Chapter.

4.2 The deck

The deck is quite thin considering that the span to depth ratio can be of the order of L/250, where L is the main span length. The low depth to span ratio is made possible because of the closely spaced (6 to 12 m) stay cables supporting the deck which provide to it elastic supports at their junction with the deck. The deflections of the deck due to applied loading are kept under control by the presence of the stay cables and are not governed to any significant degree by its stiffness. Thus, a thin deck suffices provided that, it is safe under aerodynamic and seismic effects as well as torsional loading. The deck can be constructed in situ or constituted from precast concrete segments (**Chapter - 3**). The deck may also be in composite steel-concrete construction **Fig. 4.8**, Signature Bridge; **Fig. 4.9 (a)** (Vidyasagar Setu) or structural steel. Free cantilevering technique remains the predominant construction **Fig. 4.9 (b)**.



Fig. 4.8: Signature Bridge



Fig. 4.9 (a) Vidya Sagar Setu over Hooghly, Kolkata (Main Span 457 m)



Fig. 4.9 (b) Vidya Sagar Setu over Hooghly, Kolkata (Main Span 457 m). Free cantilevering of the deck

The form of the section of the deck depends largely on the mode of support provided by the stay cables (**Chapter-3**). If the cables support the deck at its two extremities **Fig. 4.9 (a)** Vidyasagar Setu, **4.10 (a), 4.10 (b), 4.10 (c)** Naini Bridge, **Fig. 4.11** Atal Setu, **Fig. 4.12 (a), 4.12 (b)** Signature Bridge by two planes of stays, the section can consist of a simple slab-like structure with or without a grid of beams below it. It is usual to think in terms of a deck without the necessity of providing torsional stiffness as eccentric loads in the transverse sense would be transferred directly to the two planes of stay cables increasing the tension in one plane of stays and reducing tension in the other plane of stays **Fig. 4.10 (b), 4.10 (c)** Naini Bridge.



Fig. 4.10 (a) Naini Bridge



Fig. 4.10 (b) Naini: Naini Bridge Across River Ganga
Deck Width 26.4 m, Central Span 260 m

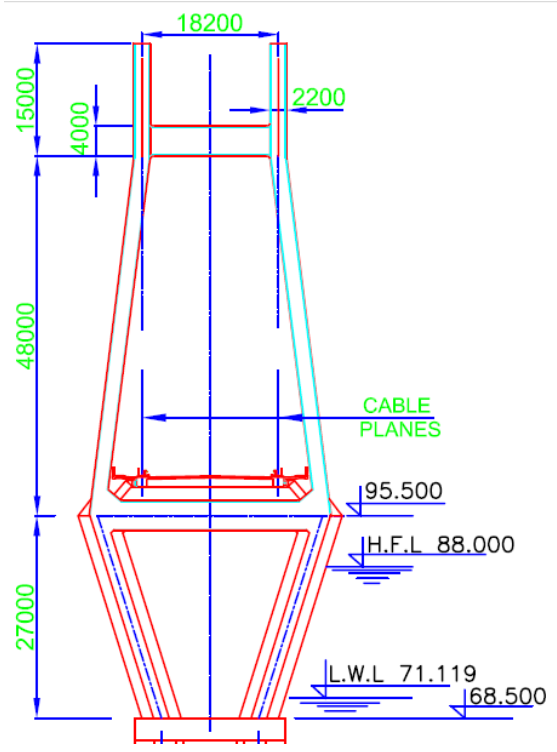


Fig. 4.10 (c) Naini Bridge

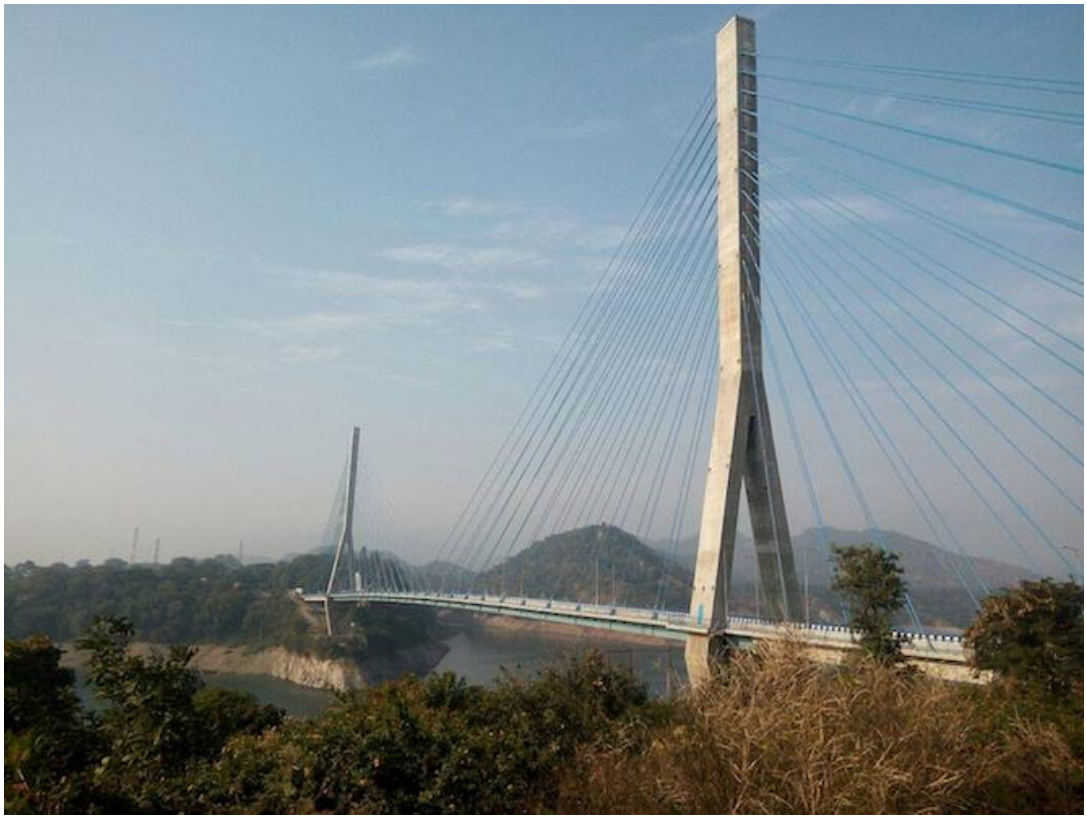


Fig. 4.11. Atal Setu over Ravi, J&K (Main Span 350 m)



Fig. 4.12 (a) Steel Grillage For Deck (35.2 m wide): Signature Bridge Before Concrete Deck Placement

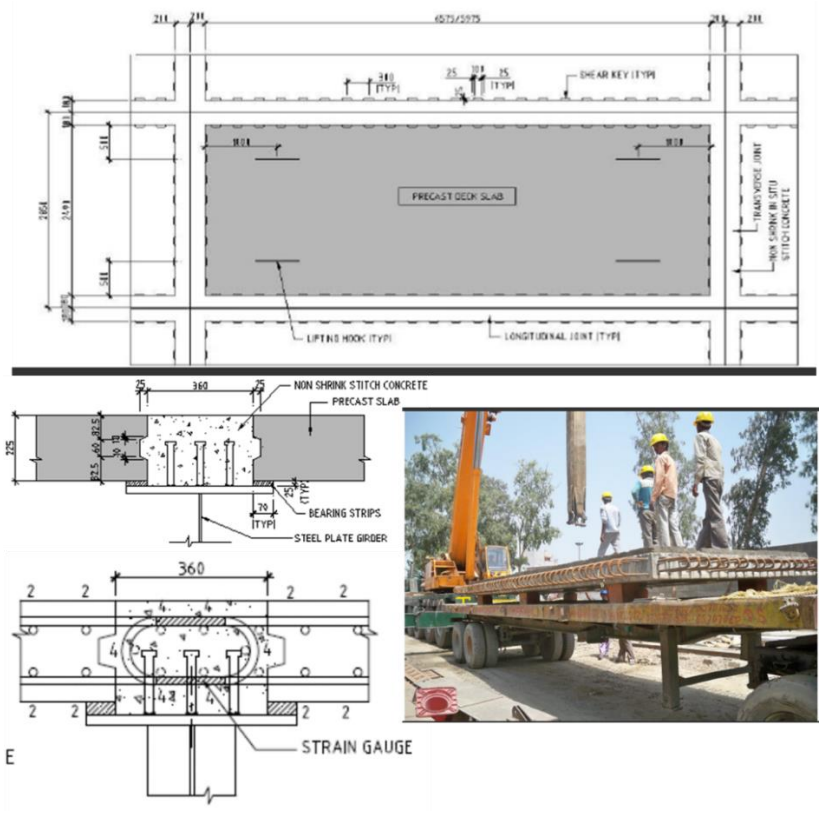


Fig. 4.12 (b) Composite Deck Structure: Signature Bridge

When a single plane of stay cables are provided at the centerline of the deck, **Fig. 4.5** (Chambal) the stay cable system will assist in transferring vertical loads only, while eccentric loads on the deck (which cannot be handled by the single plane of stay cables) a box girder section capable of resisting torsion becomes necessary. For longer span applications though, an aerodynamically designed box section is generally deployed, (**Chapter 9**).

In both the cases discussed above, i.e. single or two planes of stay cables, the deck section must be capable of spanning between the supports provided by the stay cables. Additionally, the deck must have the capability to resist compression caused by the inclination of the so-called 'self-anchored' stays, **Fig. 4.2**.

Second order effects (P-Delta) become important when the deck is thin. The calculations of the second order moments are complex due to the presence of stay cables which provide discrete elastic supports to the deck, **Fig. 4.2**, and the fact that the axial compressions get accumulated and are at a maximum near the Pylons. Svensson in **Chapter 3.1 & 3.2** of his book **[1]** explains how a simplification can be made by considering the deck as a 'beam on elastic foundation' with a constant subgrade modulus subjected to a constant axial force over the whole length.

4.3 The Pylon

The main purpose of the pylon is to transfer the loads imposed on it by the stay cables down to the top of the foundation. It also has to restrain the deck for lateral loads in the transverse direction. Since the vertical loads of the deck are catered for by the stay cables there is no need for the deck to be supported directly on the deck, **Fig. 4.2**. When a single plane of stays supports the deck the pylon is located on the median verge of the deck, ie, in the plane of the stays, **Fig. 4.13** (Chambal). The pylon is a straight-forward column cantilevering out from the foundation. However, for a laterally supported deck with two planes of stay cables the shape becomes complicated **Fig. 4.10** (Naini Bridge), **Fig. 4.11** (Atal Bridge). Being a compression member, it is almost always in concrete unless there is a special reason to make it in steel.

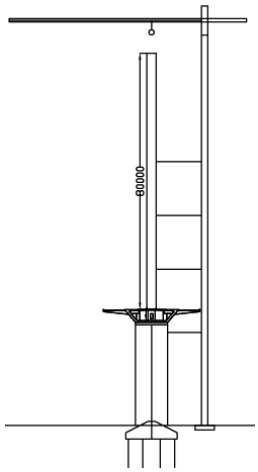


Fig. 4.13. Chambal Bridge Pylon: 80 m Tall Pylon With Single Plane of Stays Erection by Crane

The ratio of the pylon height above the bridge deck to the main span length is typically of the order of 1:5. This can be derived from the consideration that the stay cables whose main purpose is to carry vertical load are not made flatter than say 27° as otherwise they would become practically ineffective for the main purpose for which they have been provided. Incidentally, flatter stays can be provided, in which case they would act more like ‘external prestress’. Such bridges are called Extradosed Bridges in which the intention is to carry only some part of the vertical loads by stays while the remaining would be carried by the deck in flexure.

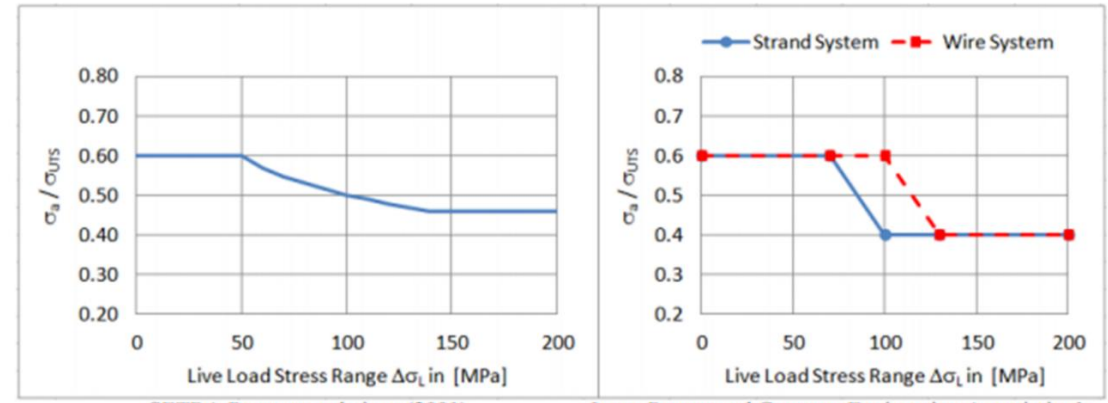
Due to the great height of the pylon its buckling and second order effects need to be paid special attention and so also the wind effects (**Chapter 9**). The stiffness of the pylon is an important consideration for reducing the sagging effects in the main span. As already discussed, the stiffness of the pylon in a bridge with only one pylon which supports one span each on either side is exceedingly important and an ” A” shape becomes appropriate **Fig. 4.7** Worli Bandra.

4.4 The Stay Cables

4.4.1 *Fatigue Strength*

The stay cables which support the deck are anchored to the deck as well as to the pylon at different heights. These are made up of high tensile steel wire. Modern day bridges use the parallel strands or parallel wires **(chapter 9),** which offer the advantage of greater extensional modulus compared to twisted wire strands or ‘locked coil’ ropes, and, thus reduced girder deflections and flexure. Cables can have a “harp” or “fan” arrangement or they can be an “intermediate” system as already described in **Chapter 3**. The spacing of cables has a profound overall influence upon the design of the bridge. A small spacing, like the one used in modern day cable stayed bridges, leads to a lighter deck girder. Furthermore, this choice results in the cable sizes being smaller and more easily manageable in transportation as well as in construction.

The design of Stay cables are primarily based on the criteria of fatigue. The axial fatigue strength of stay cables is deemed to be acceptable at 2 million cycles for usual cases. Fatigue is dependent on the change of stress in stays resulting from the application of live loads and the permissible upper limits i.e. 50% UTS for Cable Stayed Bridges and 60% UTS for Extradosed Bridge cables as specified in fib 89 and slightly different values for SETRA and the Japanese Codes as shown in **Fig. 4.14**. As can be seen, it is the change in stress in the stay cable due to live load which decides the permissible stress in the stay as a proportion of its ultimate load capacity. This fact can be advantageously used to determine the sectional area required in each stay cable individually instead of having a fixed value for the whole bridge, **Fig. 4.23**.



SETRA Recommendations (2001)

Japan Prestressed Concrete Engineering Association's Specifications [Ogawa and Kasuga 1998] [Kasuga 2006]

Stress range due to live loads decides fatigue

CABLE STAY BRIDGE ~ 0.4 Ultimate Strength
EXTRADOSED BRIDGE ~0.6 Ultimate Strength

Fig. 4.14 Fatigue of Stay Cables is the Determining Factor for Allowable Stresses

4.4.2 *The Back Stay Cable*

For balancing the permanent loads it is advisable that the horizontal component of the stay forces in the main span are in equilibrium with the horizontal component of the stay forces in the side span thereby avoiding bending moments in the pylons.

Live Loads on the main span in a typical 3 span configuration of the bridge **Fig. 4.3** the pylon is pulled towards the main span by the stays supporting it. These forces are counteracted by the tension in the stays connected to the side spans. The back stay cable is unique in the sense that it is connected to the deck at a point where there is no vertical deflection because of the anchor or hold-down piers that prevent uplifting of the deck.

The behavior of the Back Stay Cable with the Live Load in the Main Span and Side Spans respectively is shown in **Fig. 4.15**. It has the largest variation of stress amongst all stays as tension has the highest value in the first case while the second case induces compression.

The largest horizontal component inducing compression in the deck also is due to the backstay cables, **Fig. 4.15**.

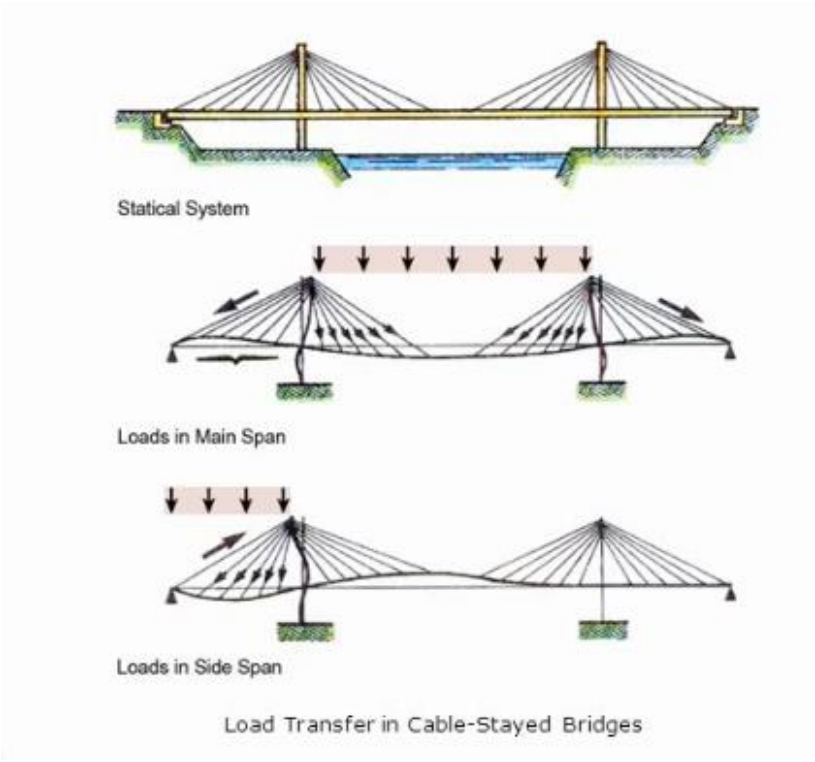


Fig. 4.15 Behavior and Role of Backstay Cable SVENSSON

4.4.3 *Seismic and Wind Effects*

Seismic and aerodynamics have contradictory demands on the structure. For seismic design a more flexible bridge will have less response. On the other hand, for aerodynamic stability a stiffer structure is preferable. Some compromise between these two demands is required. A device that connects the girder and the tower, which can break at a certain predetermined force will help in both events. Under aerodynamic actions, it will suppress the onset of the vibrations as the connection makes the structure stiffer. Under seismic load, the connection breaks at the predetermined load and the structure becomes more flexible, thus reducing the fundamental frequency of the bridge. Detailed coverage of these concepts are available in **Chapters 8 and 9**.

4.5 Steps for Preliminary Design

Preliminary design of a cable stayed bridge is essentially an iterative process**,** just as it is for any structure. The number of cycles of iteration depend upon the past experience of the designer, and, the back-up information available to him through standards and other related literature. As a first step, the general proportions of the structure relative to the span are to be assigned on the basis of guidelines discussed in the earlier part of this chapter. Next, the sizes of the various elements have to be assumed (estimated). It is clear that all the three major elements of a cable stayed bridge, namely, the deck, pylons and cables interact with each other in this highly indeterminate system, and cannot therefore be designed independently of each other. In this case nevertheless, one may begin with the deck and the cables. A good strategy is to take starting values based on data from similar (or nearly similar) existing bridges.

Since the stay cables are closely spaced, the central idea is that the deck should remain horizontal for permanent loads. The bridge deck can then be designed such that it behaves, like a continuous beam supported on rigid supports, the location of which are at the cable anchorage. Not only are the Bending moments in the deck minimized due to permanent loads but so also the creep effects due to bending of the deck. Of course the creep effects due to the axial compression induced by the stay cables would still take place.

The steps recommended are as follows:

Step 1: Select span arrangement, bridge type and configuration

Step 2: Select Cable Arrangement Type; one plane /two plane of stays (several examples shown in **Fig. 4.5** and **Fig. 4.10**

Step 3: Select appropriate Section of the deck (one plane stays will generally require box girder section **Fig. 4.5** and estimate average depth of concrete for the section **Fig. 4.21 (a).** Calculate Sectional Properties

Step 4: Evaluate first estimate of total stay quantity by assuming the value of β **Fig. 4.23** and spacing and thereby designate the area supported by each stay cable

Step 5 : Calculate DL effects in service condition

Step 6 : Calculate Live load effects in service condition..

*Step 7: R*efine the value β and re-do the steps 5 and 6 if necessary. This completes the vertical load effects on the bridge.

Step 8: With the structural model so determined one can get into Seismic and Wind Effects if so required in the Preliminary Design stage. Further processing where necessary at the Preliminary Design stage would involve partial load factors and load combinations (in accordance with IRC:6 or as modified in this document) since no provisions exist in the country specifically for Cable Stayed Bridges. Other Chapters in this document may be referred for an exposition of these ideas.

4.6 Preliminary Dimensioning and Estimation of Quantities

4.6.1 *Concrete Deck*

Mermigas **[12]** has published an interesting paper on the quantities of concrete (m^3/ m^2, or average thickness of concrete in the deck) for Cantilever, Extradosed and Cable Stayed Bridges based on previous projects. The information is shown in a concise manner in **Fig. 4.21 (a) & 4.21 (b)** and can be used conveniently in the Concept Design Stage as a starting point for self weight of the deck concrete.

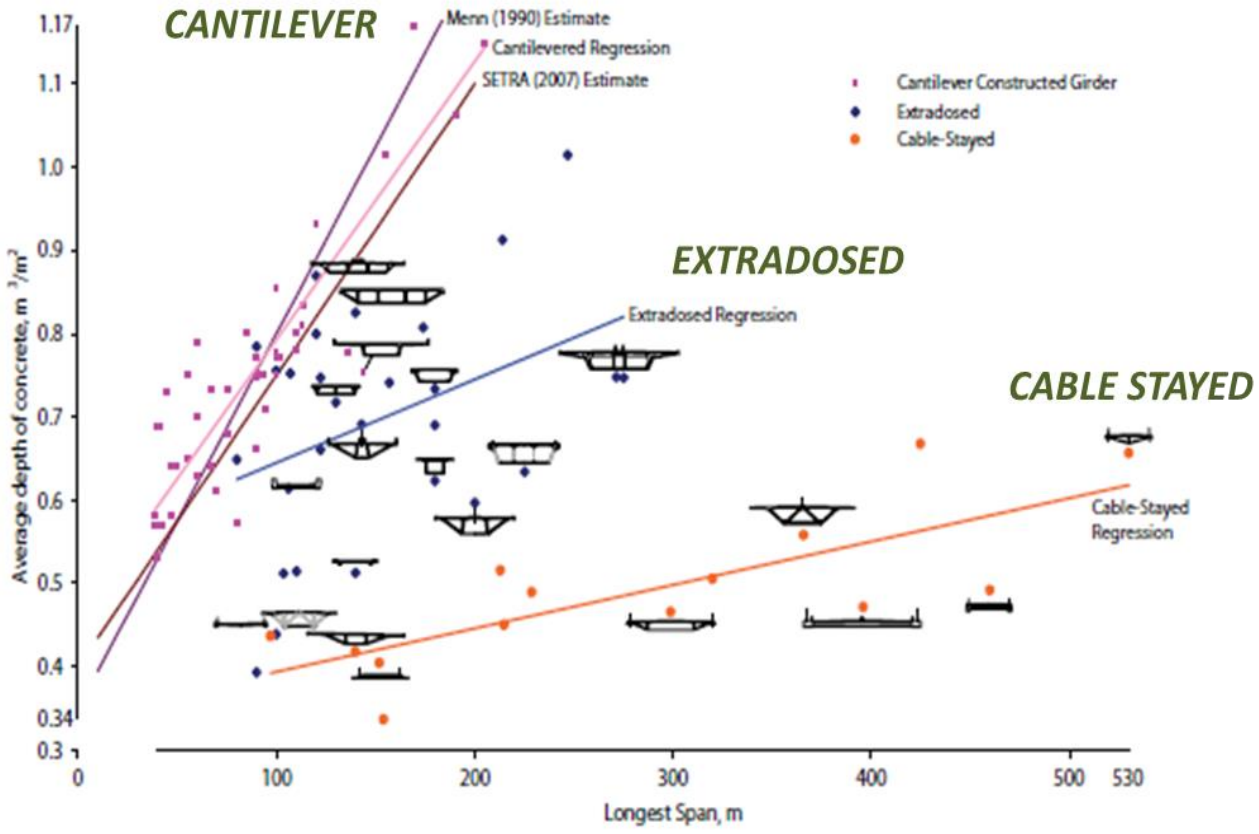


Fig 4.21 (a) Evaluating Approx V.G. Average Concrete thickness of Deck. Mermigas [13]

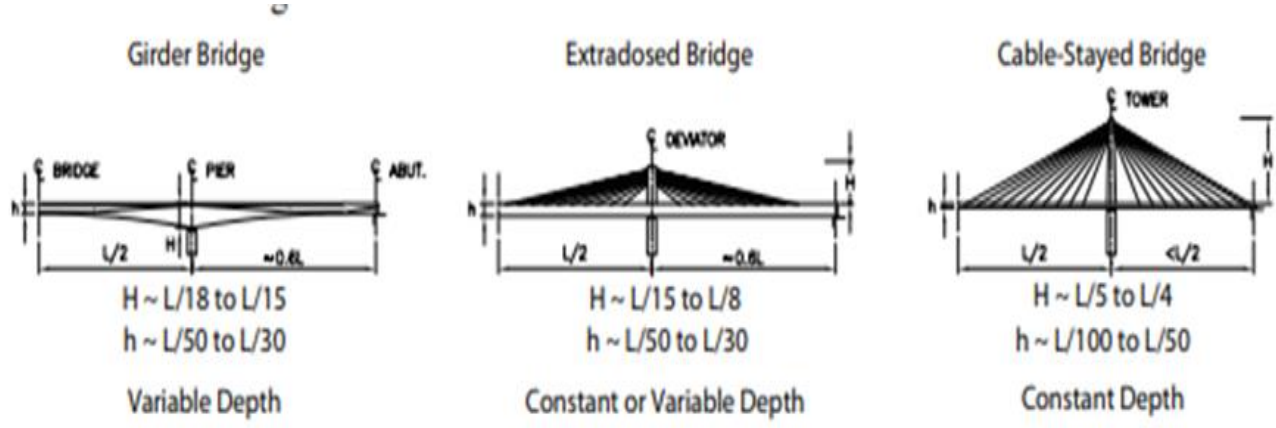


Fig 4.21 (b) Dimensioning the Superstructure of the Three Types of Bridges

As an illustration the Naini Bridge with 630 m (c/c of Expansion Joints) long module containing the main span of 260 m had an average thickness of 0.62 m including the thickened portion of the deck to prevent uplift at the bearings of the end pier as indicated in **Fig. 4.22 (a) & 4.22 (b).** To increase the factor of safety against uplift the weight of the approach span was also mobilized. The arrangement of stays with respect to the pylon was symmetrical.

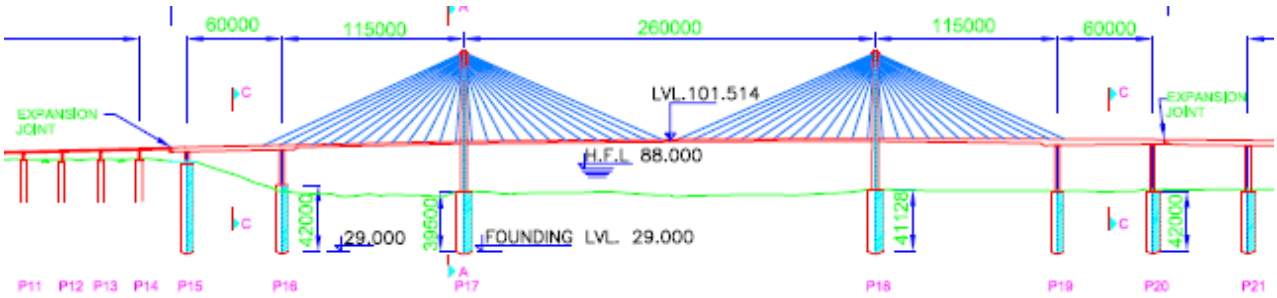


Fig. 4.22 (a) Naini Bridge Module with 260 m Main Span

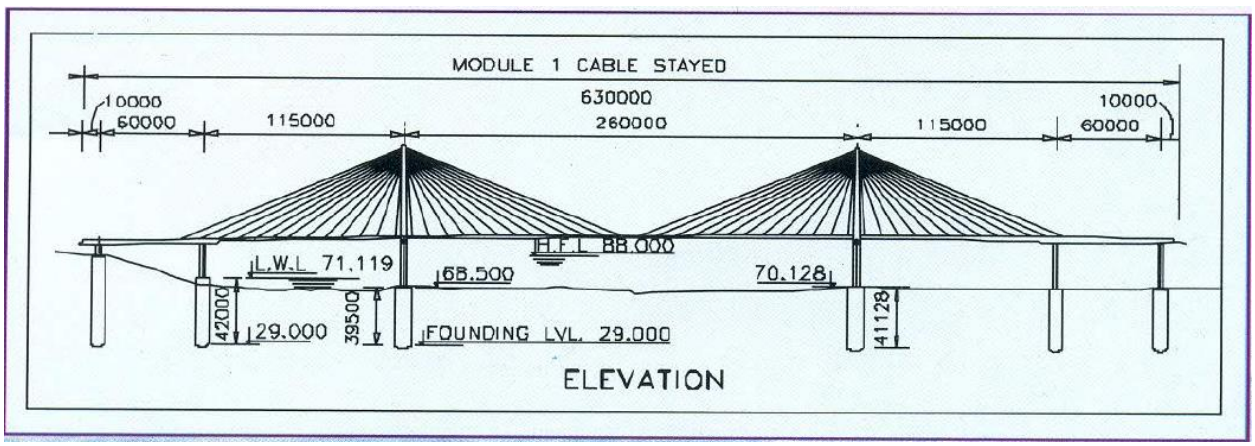


Fig 4.22 (b) Naini Bridge: General Arrangement.
Note that Reactions of Approach Spans act at the two ends of Module 1 which are also the locations of the expansion joints.

The 630 m long continuous bridge deck has fixed bearings on pier P I5 and one directional sliding bearings on the piers P 16, P 19 and P 20. The bridge deck is not supported on the lower cross beam of the pylons P 17, P 18. Only the transverse horizontal forces can be transferred directly from the bridge deck to the pylons. Expansion Joint capacity at P 20A is 1200 mm (Modular Strip Seal with 15 nos. modules)

4.6.2 *Stay Cables*

In a recent publication **[10]** Akio Kasuga has simplified the quantity estimation of stay cables for Extradosed Bridges and Cable Stayed bridges (in Concrete or steel). The ideas presented below have been taken from references 10 & 11. Ogawa and Kasuga in their in 1998 **[13]** first introduced the concept of the Beta factor which is defined as follows:

$$\beta = \frac{\text{Load carried by stay cables}}{\text{Total vertical load}} \times 100\%$$

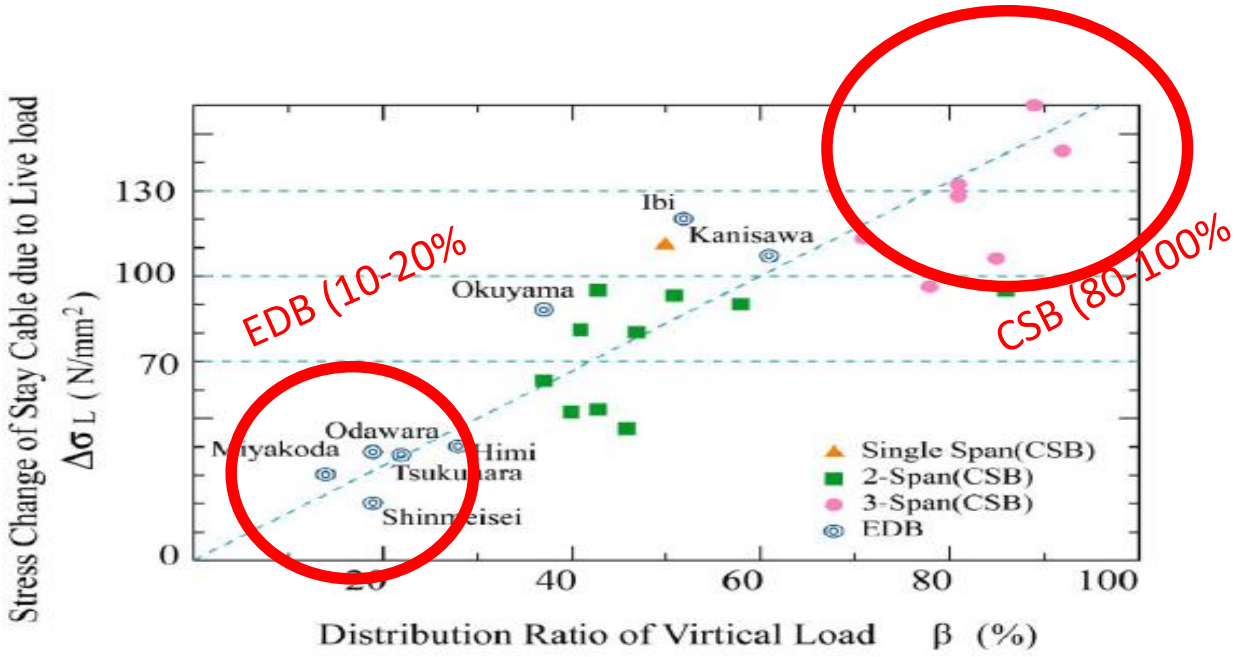


Fig. 4.23 Concept of Beta Factor applied to CSB and EDB [14]

From past projects the Cable Stayed Bridges and Extradosed Bridges generally lie in the range shown in **Fig. 4.23**. For cable-stayed bridges, β is in the region of 80 - 100%, as the cables carry nearly all the live load. In contrast, for extradosed bridges, β is in the region of 10 - 20%.

Kasuga **[10]** concludes that the cable-stayed bridge and extradosed bridge can be considered as a continuum rather than having a clear borderline between these two types of bridges, **Fig. 4.23**, with the main girder carrying most of the live load in the latter case. From this point of view the Japanese code has a great advantage as it defines the allowable stress in terms of stress variation (due to live load). It enables the designer to consider the limit separately for each individual cable. In other words, the allowable stress can differ from cable to cable.

This permits the adoption of efficient designs that were not possible under conventional standards that determined a single allowable stress for the whole bridge, be it cable stayed (concrete/steel) or extradosed. However, fib 89 **[6]** on which these Guidelines are largely based mention this idea as 'International Design Practice' without prescribing to it and also mention that such an approach would require the redefining of the limits of permissible service stresses in the cables depending on the actual levels of variable stresses.

In the recently published IABSE Bulletin 17 **[11]** the approximate range of values have been summarized for Stay cables, Extradosed cables and PT Tendons as follows:

Table 4.1 Live Load and Dynamic Characteristics of Cable Types

Cable type	Live load stress change $\Delta\sigma_L$ (MPa)	Approx max allowable stress
Stay cables	~100 ($60 < \Delta\sigma_L < 160$)	40–45% GUTS
Extradosed cables	~50 ($30 < \Delta\sigma_L < 100$)	Dependent on $\Delta\sigma_L$
PT tendons	~15 MPa	70–75% GUTS

4.7 Multi span cable stayed bridges

Multi span cable stayed bridges have to be dimensioned a little differently from the 3-span construction which largely formed the basis of discussion so far.

The tower height should be chosen so that the flattest cables have the usual inclination of about 27°. This gives a ratio of tower height to each main span of about 0.5.

If the spacing of the towers is kept constant, **Fig. 4.24 (a)** the permanent loads will be balanced which is of great benefit. However, the live loads create larger effects as compared to the 3-span construction due to the absence of short side spans, **Fig. 4.25**.

The Mandovi Bridge in Goa, **Fig. 4.24 (a) & 4.24 (b)** is currently the only multispan cable stayed bridge in the country, with a span arrangement of 85+3x150+85 m, making a total length = 620 m between abutments where the expansion joints are located. **Fig 4.24 (c)** shows the Cross Section with a total width of Bridge Deck as 21 m for 4 lanes of traffic plus 3 m median for accommodating the stay cables.

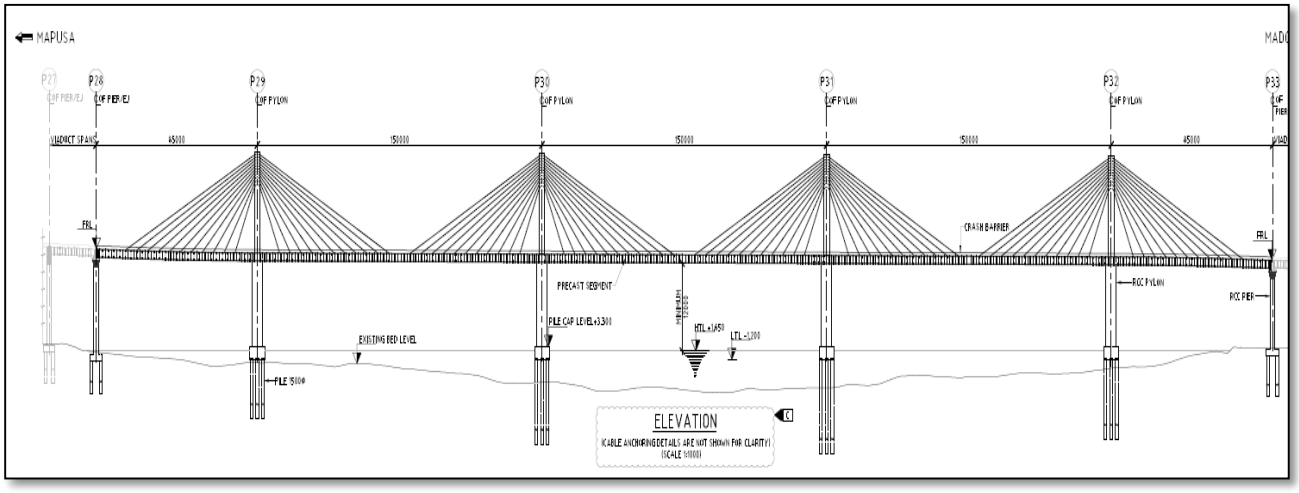


Fig. 4.24 (a) Mandovi Bridge, Goa



Fig. 4.24 (b) Mandovi Bridge, Goa Third River Crossing

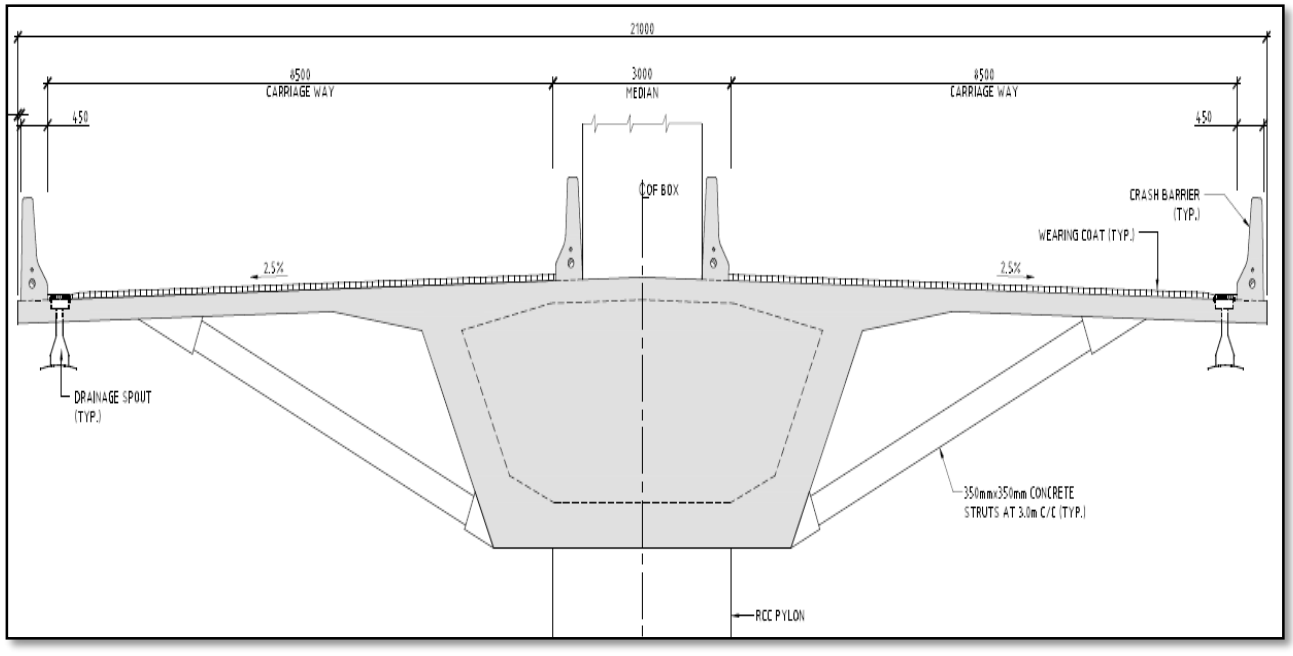


Fig. 4.24 (c) Mandovi Bridge, Goa. Deck Cross Section

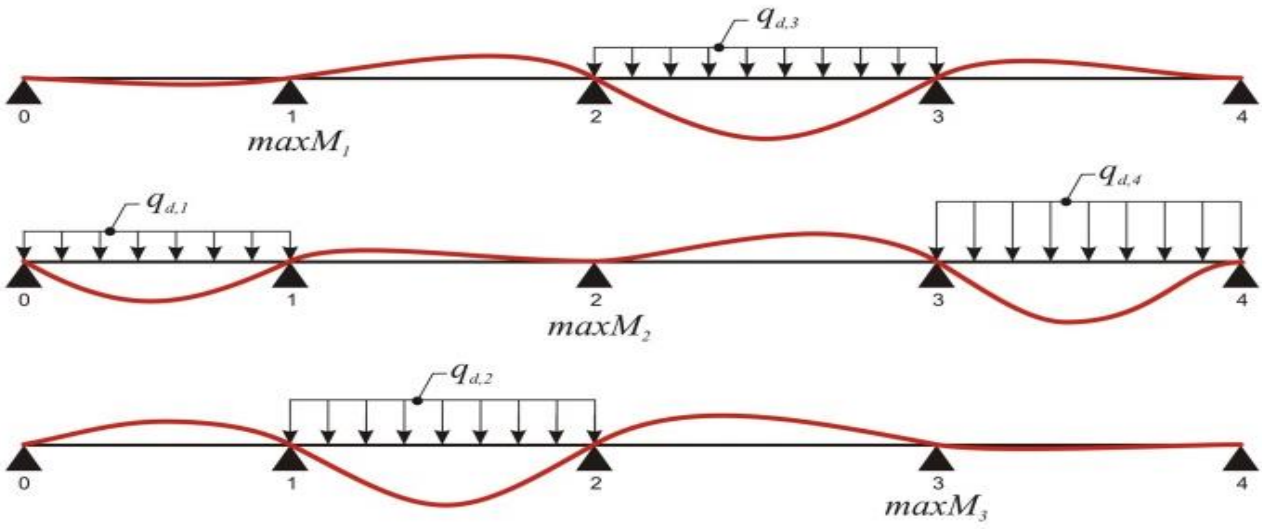


Fig. 4.25 Live Load Effects in Multispan Bridge

4.7.1 *Stabilising the pylon*

In the absence of short side spans when the stay forces on the pylon cannot be balanced, a good option is shown in **Fig. 4.26** Anchor cables from the top of one pylon are attached to the deck at the next pylon location which provides the necessary restraint to the pylon top.



Fig. 4.26 Tin KAU Bridge, Hongkong - stabilising the pylon Concept; Schlaich Bergerman & Partners

4.8 Record Breaking Spans Under Construction Bridges

Two interesting bridges under construction in India with unique concepts using steel-concrete composite construction require discussion to make this Chapter complete:

- The New Zuari Bridge in Goa (Main Span = 360 m)
- The Beyt Dwarka Bridge connecting Okha to Beyt Dwarka in Gujarat (Main Span = 500 m)

Brief details of the concepts of the two bridges are discussed below:

4.8.1 *The New Zuari Bridge in Goa*

The bridge is being constructed with a cable stayed module of 640 m with span arrangement of 140+360+140 m, **Fig. 4.27** When completed, it will be the second longest span in India after the Vidyasagar Setu, **Fig. 4.9**.

As a matter of fact there are two parallel bridges each catering to 4 lanes of traffic with a width of 27 m The bridge will cater to the heavy vehicle traffic between Panjim and Goa which can not move over the existing bridge which is in a distressed condition.

The pylons reach a height of 100 m and are planned to support a spectacular tourist observatory gallery and restaurant, **Fig. 4.28** which are expected to be a huge tourist attraction.

The deck is fabricated on shore in segments typically of size - 27 m * 12 m weighing 100 t **Fig.4.29** which are transported on barges and raised in position with the help of segment lifters used for the free cantilevering of the bridge shown in **Fig. 4.30**.

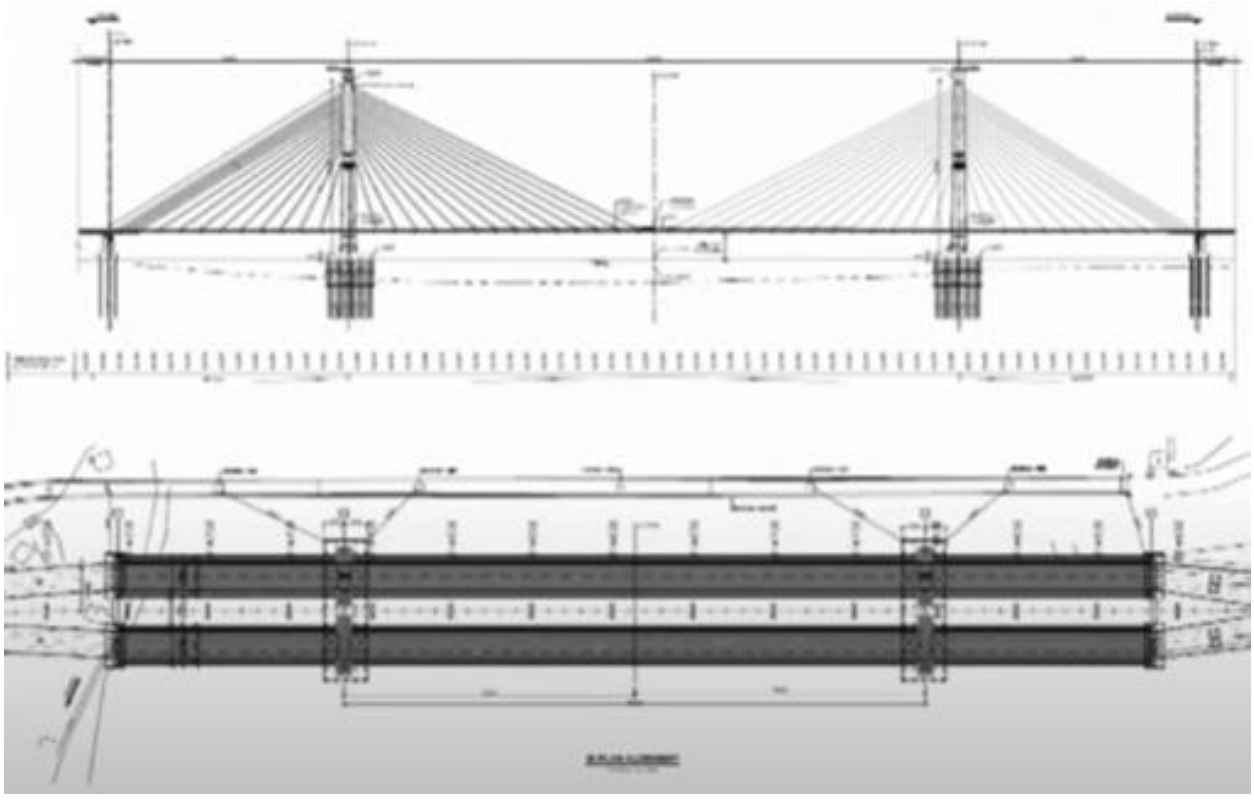


Fig. 4.27 Elevation and Plan of Zuari Bridge



Fig. 4.28 Proposed Viewing Gallery



Fig. 4.29 Steel Deck Segments Fabricated on Shore



Fig. 4.30 Free Cantilevering of the bridge for the two Decks (Constructed Simultaneously)

4.8.2 *Beyt Dwarka Bridge, Gujarat*

The bridge is proposed with a cable stayed module of 900 m having a span arrangement of 2*50+100+500+100+2*50 **Fig. 4.31**. The deck is constituted from a steel grillage with concrete deck. The approaches and the end spans are constructed first and then the main span is cantilevered out from the pylons.

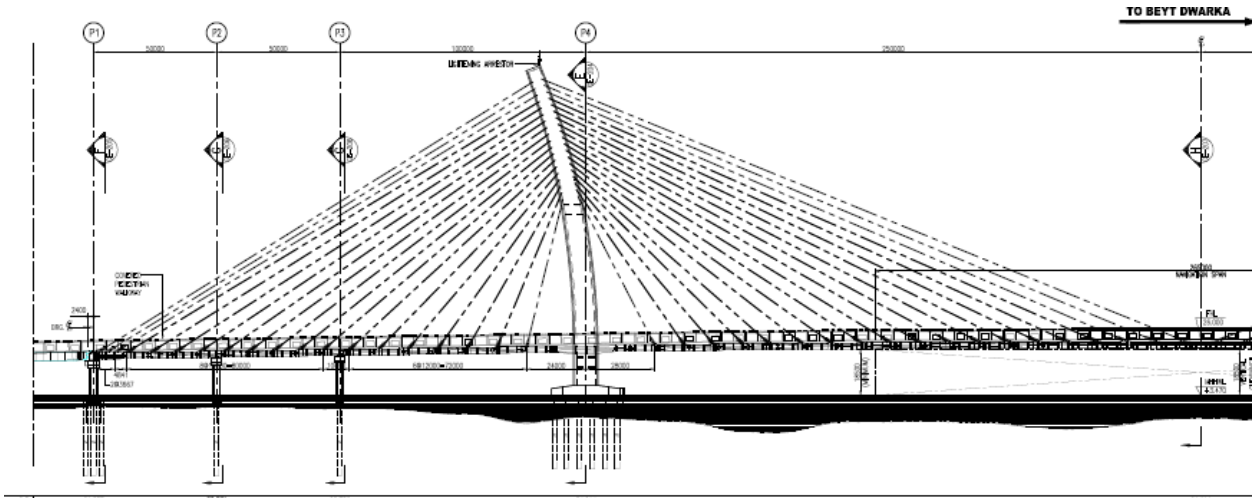


Fig. 4.31 Beyt Dwarka Bridge, Gujarat: will have a Main Span of 500 m

CHAPTER - 5
CONSTRUCTION

5.1 General

The cable-stayed bridges covered by this guideline differ from girder bridges & extradosed bridges. However, there are some commonalities between cable-stayed and extradosed bridges, like use of cables, construction of upper pylon, installation and anchoring of cables. Despite these commonalities, as the size of cable stay bridges (Span, pylon heights etc.) are comparatively larger, stress and deformation produced in the members can be significantly influenced by the construction method and sequence adopted

With regards to construction, it is necessary to satisfy required conditions such as environmental effects, construction safety, cost and period, which are based on its relation to entirety, purpose, type, scale and characteristics of project.

The construction planning of cable-stayed bridges shall be prepared prior to the commencement of construction by considering the aspects, that are necessary to satisfy the required performance of structure and requirement of the project.

In addition, aspects about the quality and schedule control also should be examined while planning construction.

In comparison to other bridges, for cable stayed bridges, the influence of construction method, construction sequence and construction accuracy are significant. Therefore, it is important that qualified engineer(s) with sufficient knowledge and experience are constantly stationed at the construction site to provide appropriate construction management.

5.2 Deck Erection and Fabrication

Generally there are three methods of deck erection for cable stay bridges. The staging method of erection is most often used where there is a low clearance requirement to the underside of the structure and temporary bents will not interfere with any traffic below the bridge. Its advantage is accuracy in maintaining required geometry and gradient and its relatively low cost.

The push-launching technique has been used successfully where care must be taken not to interfere with traffic below the bridge and where cantilever construction is impractical. In this method, large sections of bridge deck are pushed out over the piers on rollers or sliding teflon bearings. The deck is pushed out from both abutments toward the center, or, in some instances, from one abutment all the way to the other abutment in which case it is called progressive placement method. Assembling the components in an erection yard at one or both ends of the structure and progressively pushing the components out into the span as they are completed, can simplify construction and reduce costs. With this method as much as 1500 tons of steel, spanning a number of supports, have been pushed out and, in some instances, the method has been used where a horizontal curvature is required.

The cantilever erection method is very often employed in cable-stayed bridge construction where temporary supports are uneconomical or not applicable. In some cases, it may increase the steel requirements in terms of temporary post-tensioning bars, permanent pre-stressing or steel deck quantity over that required for final positioning to accommodate the increased moments and shears

during the erection process. The principal advantage is that it does not interfere with traffic below the bridge.

5.2.1 *Staging method*

In cable stayed bridges due to the inclination of cables, the load-carrying capacity of the single cable depends on the transfer of the horizontal component of the cable force through the deck. Therefore, a given cable cannot be installed before the adjoining part of the deck has been erected. Then the easiest way of erection is to support the deck over temporary supports before the cables are stressed as shown in the **Fig.5.1**.



Fig. 5.1: Deck Erection by Staging Method (Source: JV of Gammon-Cidade - Tensaccai)

The erection of cable stay deck by staging method is realized in following 4 steps:

- ✓ Erection of the deck on the permanent piers and the temporary supports. In this stage the erection is actually a normal girder erection and any of the procedures used for the construction of girder bridges can consequently be applied.

- ✓ Erection of the pylons from the completed deck, serving as a platform.

- ✓ Installation of the stay cables. In this stage the cables only need to be tensioned moderately as the final cable tension will be induced in the last stage.

- ✓ After installation of all stay cables, the temporary supports are removed and the load transferred to the cable system. During this process the deck will deflect downwards and it is therefore necessary to initially erect the deck in an elevated position to arrive at the desired final geometry when the total dead load of the deck has been transferred to the stay cables.

With the deck erected on temporary supports and subsequently lowered, a permanent bending will be induced in the deck sections. With slender decks the bending moments might well be within acceptable limits, but with a stiffer deck the procedure might have to be modified (to eliminate this permanent bending). In that case the deck must initially be erected with its final elevation on the temporary supports and then be jacked up before the stay cables are attached and partly stressed.

The erection procedure illustrated in **Fig. 5.1** offers the advantage that the deck can be erected continuously from one end to the other and thereby allows transportation of men, equipment, and

material on the completed part of the deck. Also, the procedure leads to an efficient control of the geometry and the cable tension.

The disadvantage of the procedure is related to the temporary supports that are required. In many cases a clearance profile will have to be respected also during the construction period so that temporary supports cannot be allowed. Even if the clearance requirements are not prohibitive, the cost of erecting the temporary supports, e.g. on large water depths in the main span, might be of such a magnitude that the procedure will not be feasible.

5.2.2 *Push-launching method*

The push launching method is sporadically used for girder bridges, and, as such the concept is not entirely new for cable-stayed bridges. In this method of erection, a large area behind the abutment is utilized as the assembly shop. The length of the area required behind the abutment must be larger than the side span of cable stayed bridge. The cable stays can be laid out on the deck next to the pylon and erected along with the saddles to the top of the pylon by a crane. Then the assembled span along with the pylon and cables is pushed out, cantilevering from the abutment to the predetermined length. Then the prefabricated modules are added behind and the span is further pushed out till the front part of the span rests on permanent or temporary piers. The operation continues.

The erection of cable stay deck by push launching method is realized as shown in the **Fig. 5.2**. In this respect the following is to be noted:

- ✓ In the final position the reaction load of the towers is borne by the permanent piers VIII and XI. However, during the push-out operation the tower reaction must be resisted by a lateral-beam diaphragm which in turn transmits the load to the longitudinal box girders.
- ✓ For this reason the cable stays are only partially tensioned. The jacking mechanism at the saddle is used to compensate for the cantilever deflection of the leading edge of the pushed-out section.
- ✓ When the leading edge of the bridge reaches pier VIII, the bearing is elevated to the designed height by jacks. As a result of this action the bearing pressure at pier VII is relieved.
- ✓ As the structure is pushed out farther, the bearing pressure at pier VIII will increase. Once the bearing pressure reaches its allowable limit when the leading edge extends by the designed length beyond pier IX, the bearing at pier VIII is lowered to its original position.
- ✓ This procedure is then repeated until the structure is in its final position.

The erection of the Millau Viaduct in France, **Fig. 5.3** is one of the most spectacular bridge erections ever performed by push launching method. It was especially challenging as the pre-fabricated segments could not be brought to the site and lifted vertically into place as generally occurs in cable supported bridges across waterways. Instead the super structure was push-launched from both abutments to a closing point above the Tarn River 270 m below the deck.

To give the necessary support during launching the permanent piers had to be supplemented by temporary piers at the centre of each span. The closing point between the two launched deck sections was positioned at the centre of the span above the Tarn River, where the deck is highest above the ground. Thus a temporary pier could be avoided where it would have been the longest.

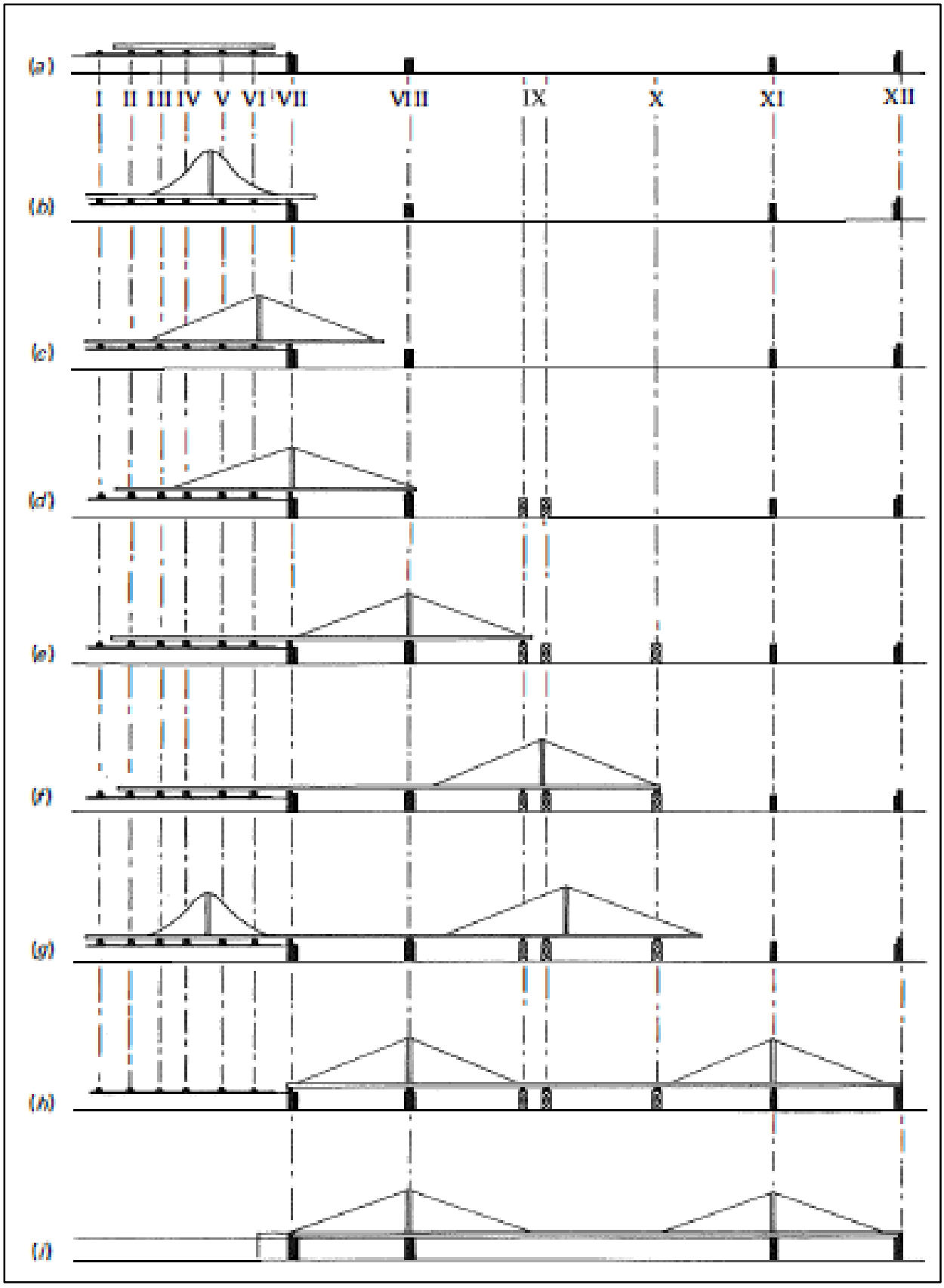


Fig. 5.2 Erection Stages for Push Launching Method [3]



Fig. 5.3 The Millau Viaduct during the launching. The deck is partly supported by the temporary supports until the remaining pylons and cable systems are added

5.2.3 *Cantilevering method of erection*

Where it is necessary to avoid temporary supports in the spans, cantilever method of construction is adopted for erection. There are two types of cantilever methods of construction:

- Balanced cantilever method of construction;
- Free cantilever method of construction.

For using this technique, construction plan shall be made taking into account the conditions of construction site, economic factors and scale of construction. Especially, it is necessary to consider cable arrangement and the sequence of prestressing. If the deck is of cast in situ construction, the concreting is done in form traveller in 3 to 5 m long segments along the axis of the bridge. In some cases, the use of special form traveler which are modified in order to avoid intersection of main frame and anchorage of stay cable, or equipped with facilities for moving back may be required. Since the cross section of girder of cable-stayed bridges is usually constant, the same formwork may be repeatedly used. Also, because the height of the girder is comparatively low and web is often slanted, materials and structure of formwork should be appropriately chosen depending on its objective. Inside the box girder, on the upper surface of the slanted web, wooden formwork attached fiber sheet can be used to prevent the formation of bubbles.

In case of cantilever erection of cable-stayed bridges, anchorage blister and cross beam which are arranged at the anchorage of stay cable are cast with the girder because of the convenience of prestressing of stay cable. There may be a situation when the formwork is needed to be assembled and disassembled every time the wagon moves. Therefore, it is necessary that structure of formwork for inside of girder is planned so that proper workability is maintained.

For precast segmental construction or steel composite deck construction, precast segments or steel girders in modules are prefabricated at the fabrication yard, transported to the location and connected to the adjoining segments already positioned in case of precast segments or steel module. Depending upon the available logistics, the precast segments or steel modules are handled by segment launcher or barge mounted cranes and held in position before being connected to adjoining segment already positioned and prestressed, or, connected by bolting to adjoining steel module already positioned.

Comparing with other girder and extradosed bridges, cable-stayed bridges have larger displacement and vibration during casting of concrete, larger vertical and longitudinal displacement by change of temperature (temperature gradient and uniform temperature) and horizontal displacement due to wind. Therefore, when constructing the closure part, if the displacement and vibration by casting concrete, change of temperature and wind are large, the countermeasures for these problems shall be implemented. The false work mechanism which can control and suppress these displacement and vibrations shall be used if necessary.

Also, consideration to cast the concrete or place the precast/steel segments during the time when change of temperature is relatively small in a day is required if the displacement by change of temperature is large.



Fig. 5.4 Balanced Cantilever Method of Erection
(Source: Left: Gammon India ltd, Right: [1])

The erection of cable stayed deck by balanced cantilever method is realized in the following steps as shown in the **Fig. 5.4**:

- ✓ The pylons (and the deck units above the main piers) are erected and fixed to the piers.
- ✓ A balanced cantilever on either side of the pier is initiated using segment lifters/derrick cranes operating on the deck to lift deck segments transported to the site on barges/trailers. The large capacity cranes can also be used from the ground/river if the ground accessible or crane can be barge mounted.
- ✓ As the cantilevers grow, the stay cables are installed and tensioned initially to relieve the bending moments in the deck. Often the cantilever process is completed for one bridge half, before moving the cranes to the other half.

- ✓ The deck is joined at midspan and additional loading from wearing surface, railings, etc., is applied.

With this procedure it is essential to have a very efficient fixity of the superstructure to the pylon piers throughout the Construction period, as the entire stability depends on this fixity until the end pier is reached. Also, the lateral flexural stiffness of the deck must be sufficient to ensure the stability of the cantilever arm that will eventually reach a length corresponding to half the main span length. Thus, the procedure is especially advantageous in bridges with a reasonably large width-to-span ratio of the deck.

The balanced cantilever method of erection is most efficient if the distance between the cable anchor points is chosen so that the deck can be cantilevered from one cable anchor point to the next without requiring a temporary support (e.g. by temporary cables). This condition is often fulfilled in bridges with multi-cable systems.

In free cantilever method of erection the side spans are erected on temporary supports and the main span by a free cantilever, as shown in **Fig. 5.5**.



Fig.5.5 Free cantilever Method of Erection [1]

The erection of cable stay deck by free cantilever method is realized in the following steps:

- ✓ Erection of the deck in the side spans using temporary supports, followed by the erection of the pylons when the main piers are reached.
- ✓ One-sided free cantilever of the main span deck with installation of both the main span and the corresponding side span stay cables as the relevant anchor points in the main span are reached.
- ✓ Completion of one half of the bridge, followed by the cantilevering of the other half.
- ✓ Closing of the bridge at the main span centre.

With this erection procedure, the application of temporary supports is limited to the side spans where clearance requirements and water depths seldom will exclude such supports. At the same time the cantilever into the main span can start from a very stable position with the side span deck supported on both the main pier and the end pier. A temporary moment-rigid fixation to the main pier is therefore not required.

5.2.4 *Temporary cable system during erection*

- Temporary erection stays can be used in cable stayed bridges with few stay cables, resulting in considerable distance between the cable supported points of the deck. In these bridges overstressing might occur either in the deck section or in the last erected stay cable from which

free cantilevering is being done. In such a case a temporary erection stay, as shown in **Fig. 5.6** (Left), might be installed to reduce both the bending moments in the deck and the tension in the permanent stay cable.



Fig.5.6 Temporary stays in the pylon (Source: Left: L&T, Right: [1])

As an alternative arrangement, the cantilevered part of the deck may be supported by a secondary cable system comprising a temporary pylon, with a set of temporary erection stays, as illustrated in **Fig. 5.6** With this arrangement, the hogging moment in the deck is reduced, whereas the tension in the permanent stay cables remains practically unchanged. Thus, this arrangement is only applicable if the problem of overstressing is confined to the deck.

However, it may be noted that temporary measures in the form of supports, stays, and pylons used only during erection are costly as they have to be fabricated, erected, and demolished before the construction is completed. Therefore, it becomes imperative that various aspects of erection are taken into consideration in the very early design stages so that the main layout of the structure can be chosen to minimize the amount of temporary structural members.

5.2.5 *Construction Methods based on type of deck / material used for deck*

Selection of deck type in Cable-stayed bridges is key parameter in terms of project costs and construction period. Based on the deck system, feasible erection methods shall be studied and decided upon, taking into account the maximum weight and length of the element that can be lifted or cast, the material as well as construction cost in terms of equipment, facility and time, such as to lead to the most economical system.

5.2.5.1 Concrete Deck

If concrete deck is adopted in a cable stayed bridge, most feasible construction method of deck is either cast-in-place deck or precast segment deck erection. It has to be chosen based on overall length of bridge, cost of precasting facility and accessibility of transportation etc.

Precast Elements

The advantage of precast elements is their fabrication under controlled conditions, independent of adverse weather. Segment weight and length shall be decided considering the lifting capacity of derrick crane and cycle time of erection. Longer segments require heavier lifting capacity in general and may require more temporary tensioning such as Post tensioning, or, ancillary stay cables.

The precast elements can be floated to site on barges or transported through truck, and, lifted by derricks located on each tip of the already erected beam, cantilevered to both sides (of the tower **Fig. 5.7 (a) & (b).** The elements are placed in such a way that they are balanced on either side about the tower. Each derrick is tied back to erected deck temporarily.



Fig. 5.7 (a) Lifting of Precast Segment (Pasco Kennewick Bridge, Washington, [1])



Fig. 5.7 (b) Lifting of Precast Segment Cable Stayed Bridge across River Mandovi, Goa, (Source: L&T)

Cast-in-place (CIP) Concrete

Cast-in-place deck is often cast by application of a form traveller attached to the completed part of the deck, **Fig. 5.8.** With the modest depth of the edge girders generally found in these bridges, the moment capacity of the deck is quite limited, so it will be necessary to cast the deck in relatively short segments with a length of typically 4 m - 6 m and the full width of the deck. The form traveller has to be designed so that it can be moved forward and cantilevered after the newly cast concrete segment has hardened and the permanent stays are installed and tensioned. With a reasonably short segment length in each casting, it will be possible to avoid temporary cables and it may not be necessary to strengthen the edge girders for the loading that comes during construction.



Fig. 5.8 Balanced Cantilever Erection by form Traveler [1]

5.2.5.2 Steel Deck

The usual method of building the main span steel deck is free cantilevering which is one of the main advantages of cable-stayed bridges. The deck sections with a length of 10 to 20 m, equal to the cable distance, are completely preassembled and transported to site on water, if possible, or on the already installed deck. The lifting takes place either by derrick from the already erected deck or by floating crane, **Fig 5.9 (a)**. Large block erection of orthotropic steel deck (maximum length of 112.7 m and 2302 ton) using heavy lifting capacity of floating crane was carried out for the erection of back span in Incheon Bridge followed by the small block erection using the derrick crane in main span, **Fig. 5.9 (b)**



Fig. 5.9 (a) Lifting of Steel Deck [1]



Fig. 5.9 (b) Large Block Erection in Back Span (left) and Small Block Erection in Main Span(right) (Source: Incheon Bridge Company)

5.2.5.3 Steel Composite Deck Fig 5.10 (a) & (b)

Composite deck is built from a steel girder grid on which rests a concrete slab, either CIP or precast.

During free cantilevering, the grid sections are lifted by derrick and welded to the already erected beam between the cable points. Then the concrete slab is installed. The steel beam should not run ahead of the concrete slab because in that case the action forces are very difficult to determine due to shrinkage and creep.

Precast slab panels are advantageous for rapid erection and reduced shrinkage and creep values due to their advanced age at installation. One disadvantage is their joints, which are mostly in compression in both the directions under permanent loads.

More time is required for false work of the slab and placement of reinforcement in case of cast-in-place slab.



Fig. 5.10 (a) Lifting of Steel Beam and Cast-in-place Concrete [1]



Fig. 5.10 (b) Steel Composite with Cast-in-Place Concrete (Source: Basholi Cable Stayed Bridge, J&K, S P Singla)

5.3 Construction of Pylon

The methods used for the construction of pylons are generally the same as for other tall, slender structures. The pylons could be in concrete or steel. Traditionally, pylons are designed with a pronounced slenderness in the longitudinal direction of the bridge to allow the pylon to follow the longitudinal displacements of the cable system. However, during construction of the pylons, the cable system is not in place so the pylons must possess the necessary resistance against horizontal wind load by themselves. Often the most critical phase in the construction of the pylon occurs in the period from when the pylon is at full height until the first stay cables or the catwalks are erected. In this period wind-induced oscillations of the pylon in the longitudinal direction of the bridge might be experienced. These are generally caused by the slenderness of the pylon in this direction, combined with the large area of the cross beam at the top exposed to wind and the climbing crane.

The problems related to wind-induced oscillations of the free-standing pylons will generally be more pronounced for steel pylons than for concrete pylons due to the steel structure's smaller mass, larger flexibility, and lower damping. In a number of cases it has therefore been necessary to take special measures to achieve aerodynamic stability in the critical construction phases. In Japan, larger pylons often have had tuned-mass dampers installed temporarily once a certain height has been reached and until a supplementary support is provided by the cable system. For major cable stay bridges with very tall pylons it is recommended to perform wind tunnel tests to check the stability of the freestanding pylon and to determine in which phases it will be necessary to take precautionary measures.

As explained earlier, the pylon must be flexible to follow the displacement of the cable system, and as the displacement of the pylon top increases significantly with the side span length, the problems of fulfilling the requirements of stability during construction and flexibility in the final stage will be more severe in bridges with large side-to-main span ratios. If both requirements cannot be fulfilled, a temporary support of the pylon in the longitudinal direction may become a necessity.

5.3.1 *Concrete pylons*

For formwork, large wooden panel system is usually used. Wooden formwork is also generally used for the anchorages of stay cable. For saddle construction, since it has a complicated shape, pre-fabricated steel formworks can be used. To save on labor and to maintain sufficient durability, precast concrete panel may also be used as formwork which is left in place after casting.

Since sectional force in a pylon is very large, densely arranged reinforcement is inevitable, because of the pylon being a slender structure. he accuracy of reinforcement processing, assembling and connecting has large effect on the workability, structural strength and durability. Therefore, the reinforcement must be processed and arranged exactly in accordance with design documents.

If a steel frame (**Fig. 5.15**) is used, it secures the construction accuracy of the tower, and also helps in fixing the reinforcing bars and formwork. In case the saddle is used for fixing stay cables, improvement of safety and workability in the site work is required such that the steel frame can be used as a structure supporting the saddle frame. The steel frame is fabricated in blocks. The blocks are then temporarily assembled to check the fabrication accuracy such that the verticality of the connection device and anchor material of the stay cable or saddle is ensured. Then blocks are transported to construction site and assembled. It is essential to precisely arrange and set the pivot block at the construction site.



Fig 5.11 Concrete Pylon During Construction showing, Tower Crane, Hoists, Climbing Scaffolding and Falsework for Cross Beam [1]

The full scaffolding assembled around the pylon frame is the most common method used for the construction of pylon. It has a merit that it can be applicable to the complicated pylon shapes with high versatility though, assembling and disassembling are carried out at a high location and therefore maintaining the safety is a main concern. On the other hand, in case of the climbing scaffolding, **Fig. 5.11,** which is mechanically moved for hoisting, excess facility such as tower crane, elevator become necessary, but this is much better in terms of safety. The climbing scaffolding system could be either jump formed system or slip formed. Jump forming is preferred over slip forming as the former provides more flexibility for insertion of various add-ons during construction of pylons.

For constructing horizontal cross beam for pylon of cable-stayed bridges, generally the falsework is placed on steel beam supported by brackets which are fixed at tower surface. If the pylon is slanted, bending moment is produced; therefore, temporary strut or reinforcement by HTS may be required to resist the same. Also, it is necessary to plan the structure and support method of temporary strut with consideration of dismantling them.

Hoist facilities used in construction are determined by terrain conditions, shape of tower and weight of load to lift. However, tower crane is commonly used when the height of pylon or pier is high. When the height of tower crane becomes high, stay support of crane fixed to the pylon structure is needed. In this case, it is necessary to examine for the local stress generated in the pylon concrete due to the reaction of stay.

5.3.2 *Steel pylons*

For smaller steel pylons of moderate size cable stayed bridges the erection might be carried out by mobile cranes with high booms or floating cranes, **Fig.5.12**. Erection by land or sea-based cranes may also be used for at least the lower parts of high pylons. With the evolution in size of floating cranes experienced since the 1970's it has become possible to erect the entire pylon in one or two pieces.

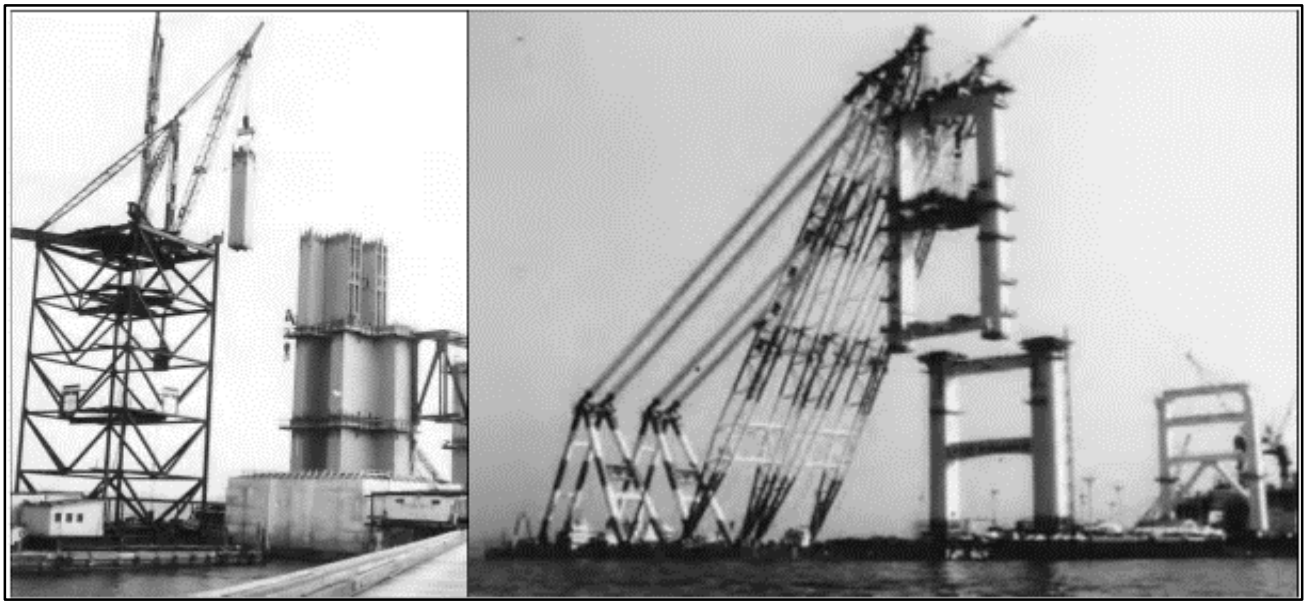


Fig.5.12 Erection by floating derrick cranes [3]

For very high pylons it is necessary to erect at least the upper part by the traditional procedure where a climbing crane follows the pylon as it grows. The crane applied to hoist the prefabricated pylon units into place generally consists of a derrick crane on a latticed cross beam attached to both pylon legs, as illustrated by the erection of the pylons for 2nd Hooghly bridge (Vidyasagar setu) at Kolkata, **Fig. 5.13 (Left**).

With the advent of very high capacity cranes, the very tall pylons may be erected by crane using prefabricated segmental construction technology. In signature bridge at Delhi, 154 m tall slanting pylon is erected segmentally by 1250 t capacity crane as illustrated in the **Fig. 5.13 (Right).** A tower crane is also connected to pylons which is attached with passenger hoist for transportation of materials, equipment for cable stressing and installation, and, crews.



Fig. 5.13 Prefabricated segmental erection of steel pylons (Source: Left: Gammon India Ltd, Right: JV of Gammon - Cidade - Tensaccai)

5.4 Construction of Saddle and Anchorage of Stay Cables

The installation position and inclination alignments of stay cable change through the duration of construction due to the changes of prestressing force and deflection of pylon and deformation of other members. However, basically, the positions during service of bridges shall be adjusted precisely in order to conform to design calculations. Therefore, in setting the saddle and anchorage of stay cables, the effects of sag of stay cable, camber when the main girder is constructed, concrete creep and shrinkage shall be considered to ensure that the saddle and anchorage will be in precise position during service of bridge.

The anchorage assembly at deck or beam level has to be properly designed depending upon the type of the deck, whether the deck is of concrete, steel or composite. The typical assembly in concrete and steel or composite deck is given in **Fig 5.14**. The design of cable anchorages should be such that if required, the assembly should enable the replacement of cables with out complexities. Generally, steel casing pipe is embedded inside the concrete member and the stay pipe including the cable is placed in the casing pipe. Where the deck is steel or composite, the anchorage assembly is welded to the girder while the bottom plate of the anchorage assembly is fitted into the grove on top of the girder to take care of the horizontal forces arising out of the inclination of the stay cable. It is imperative to carry out the special water proof and anti corrosive treatment surrounding the area where anchorage assembly meets the deck area.

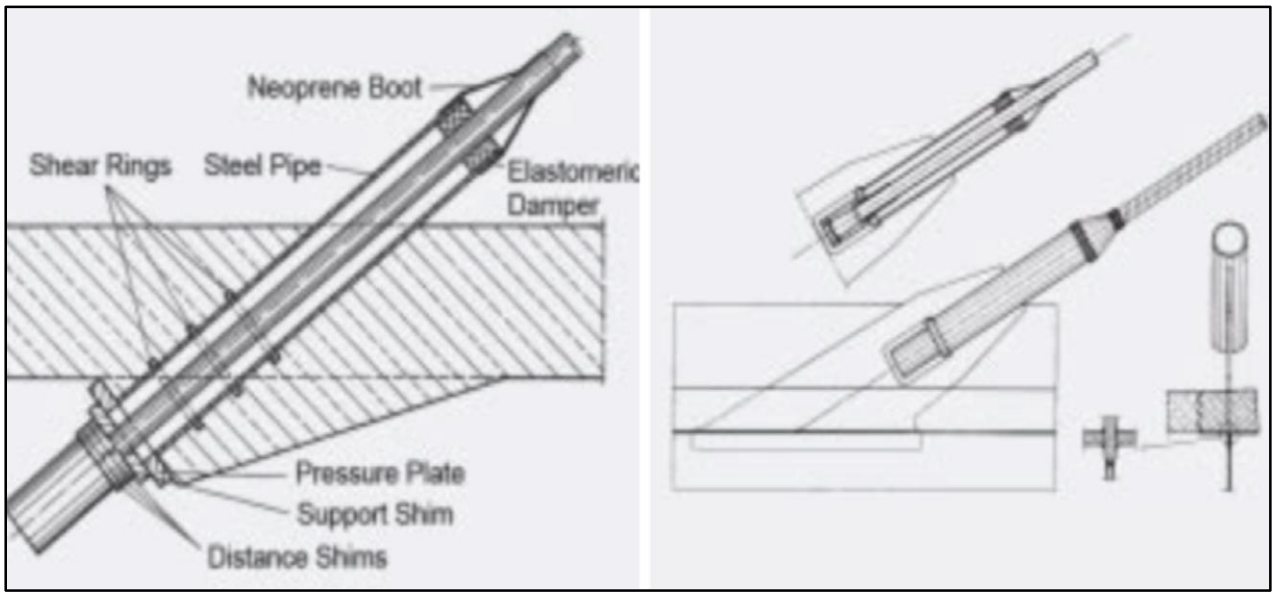


Fig. 5.14 Anchorage Assembly at Deck level for Concrete (Left) and Composite (right) girders [1]

Usually the anchorage device is fixed with the embedded steel frame, **Fig. 5.15** inside the concrete of pylon. If steel frame is used for installation of the anchorage, there are the cases of using adjustment tools, which allows micro adjustment both of up-down and left-right for cable anchorage. In order to secure the accuracy of saddle installation position, it is desirable to unitize the unit structure, as shown in **Fig. 5.15 (Right)**

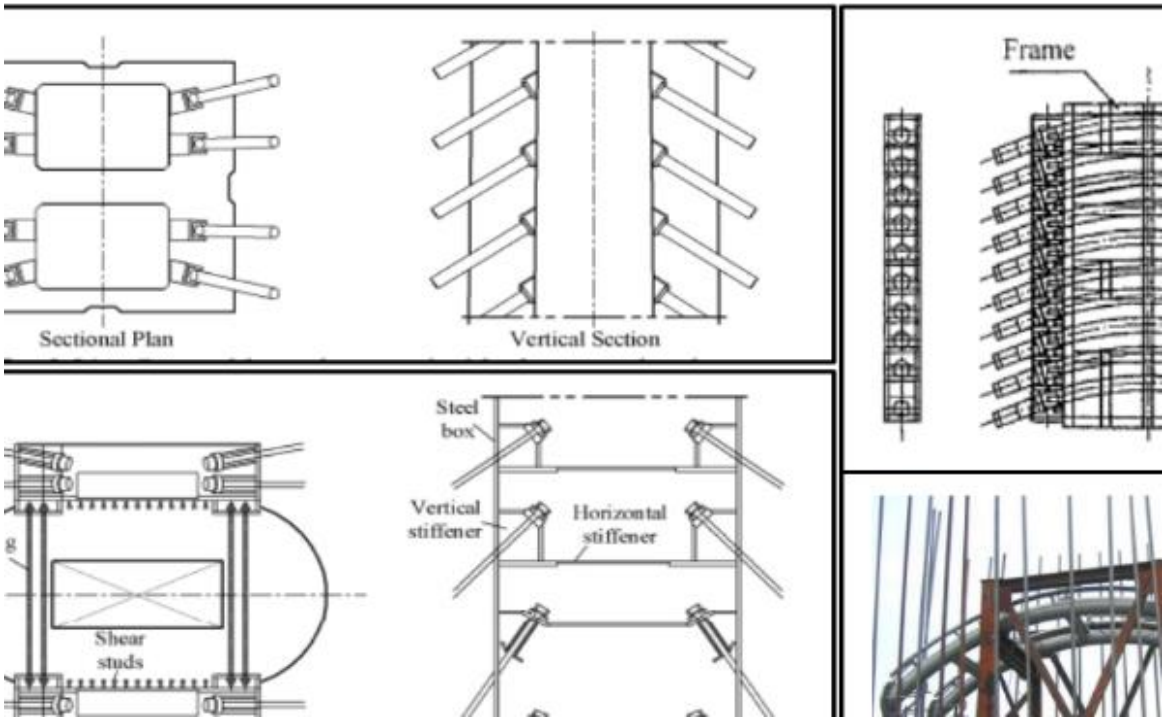


Fig. 5.15 Left : Steel Frame Inside and Outside Concrete Pylon, Right: Steel Frame for Precise Positioning of Saddles inside Concrete Pylon [2]

Special attention should be paid to ensure that no local bending of the cables occurs due to use of the saddle, which is composed of the HDPE pipe, and steel pipe, which are bent to the required shape. Therefore, it is necessary to make the pipe edge of the saddle free from local bending. In cases that the lengths of the cables are extremely long, it is also necessary to investigate a structure, which can absorb unexpected effects such as construction error and sag effect. Also, it is necessary to consider these matters with regards to not only saddle but also anchorage structures for stay cable.

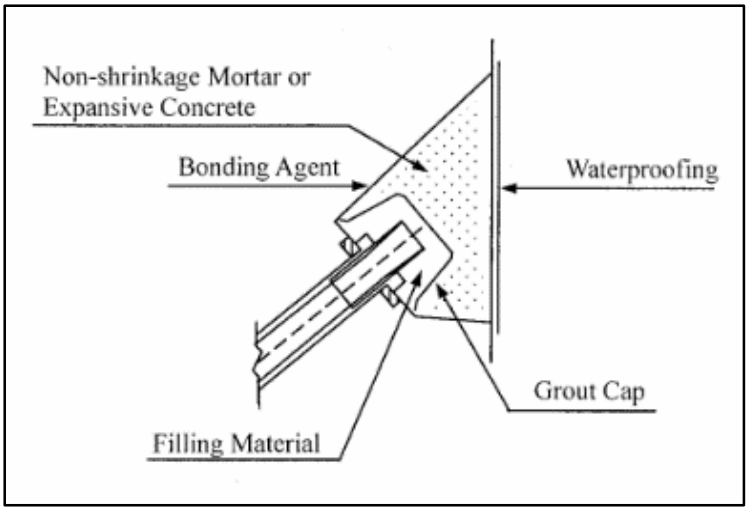


Fig.5.16 Notching Arrangement [2]

In case that the stay cables are anchored at the position which is made by cutting off a part of structure of pylon or girder, the notching configuration, **Fig. 5.16** should be determined with consideration of aesthetic. Moreover, the color adjustment of cover concrete should be considered if necessary. After prestressing, adjusting the prestressing force and grouting, the notching area shall be filled with non-shrinkage mortar or expansive concrete for being protected. Also, if necessary, providing additional reinforcement to prevent cracks on the surface, applying bonding agent and waterproofing should be done to assure rust prevention.

5.5 Construction of Stay Cable

This section describes the methods of construction of stay cables, their installation, stressing etc. for getting the general idea, and may vary from project to project depending on the contractor and supplier of the cables.

5.5.1 *Fabrication and Transportation of Stay Cables*

5.5.1.1 Generally, stay cable system consists of inclined cable (MTE), anchorage, stay pipe and filling material. Depending on whether stay cables are fabricated in a factory or at the construction site, the transportation method, placement and installation process vary.

Pre-fabricated stay cables usually fabricated as all one unit with HTS, stay pipe and filling material. Therefore, since it is difficult to adjust the length of stay cables at the construction site, it is important to determine the length of cable to be fabricated considering deflection of cable, and difference of as-build dimensions of pylon and girder from design documents. Site-fabricated stay cables can be assembled in parallel with construction of pylon and girder, and constituent material can be brought into construction site individually. The constructed area of girder or on the ground backside of abutment can be used as the space for processing the cable. However, because the

space is limited and there is congestion of different work, it is necessary to plan and create an efficiency site layout and work schedule.

The use of site-fabricated stay cables which is different from conventional system is in vogue. For example, they are the non-grout type which does not require grouting because the cable is coated with zinc or resin, and the semi-fabricated type where the coated cable, stay pipe and filling materials are integrated in the factory and carried to the site. Since existing examples of these cables are few, it is necessary to fully understand the characteristics of the materials and system prior to construction.

5.5.1.2 Pre-fabricated cables are usually transported to the site, rolled up to the reels of 2.0 m to 3.0 m diameter. The size of the reel is, of course, depends on the diameter and length of the cable. Therefore, in case of the land transportation, it is important to consider the law restriction issue in advance.

5.5.1.3 It is important for the stay cables and anchorages to be protected from damages during loading, packing, unloading and especially during storage. For site-fabricated cables, it is necessary to pay attention to rust prevention because the retention time tends to be longer.

5.5.1.4 Stay pipe is used for the purpose to ensure the durability, rust prevention and overall protection. Its material can be HDPE pipe, FRP pipe and steel pipe. Therefore, suitable materials should be chosen depending on the cable fabrication and installation method and rust prevention method. In addition, in case of the use of long cable, it is necessary to consider that the connection and erection are safely done and winches have sufficient capacity to push and pull the cables.

5.5.2 *Installation of Stay Cable*

In recent days, majority of Stay Cable Bridge Projects in India are executed by using Parallel Strand cable systems. This system is executed by site fabrication by unwinding coils of strands and stressing. Below procedure may vary as per vendors.

5.5.2.1 Preparatory Works for Cable Assembly.

- **Storage of Components**

  All components for the cables shall be stored properly to prevent contamination and damages. All parts of the anchorages, especially the wedges, anchor blocks with ring nut and the passive anchorages shall be stored at a dry place and protected against contamination. All HDPE-pipes for the sheathing shall be stored properly, lying straight without being exposed to deformations in cross direction.

- **Working Platforms**

  Working platforms outside the pylon are required for assembly of the cables up to the end of the finishing works. At the deck structure, appropriate measures have to be taken to have access to the anchorage area. Platforms and scaffoldings need to be provided by the construction company with their required minimum dimensions in consultation with the vendor.

- **Equipment**

  General equipment like mobile cranes, forklifts, power supply, etc., has to be provided.

- **Anchorages Preassembly**

  Anchorages will be preassembled on site. Part of the preassembly is also the placing of the wax plate, shown in **Fig 5.17**.



Fig. 5.17 Poured Heated wax into back of Anchorage once all holes are plugged to form the wax plate (Credits: L&T)

- **Installation of Active and Passive Anchorages**

  Prior to installation of the active anchorages, the trapezoidal thread of the anchor block that will not be accessible afterwards has to be protected, **Fig 5.18** with protection compounds:



Fig. 5.18 Filling thread with FILL and Wrapping with Petrolatum Tape (Credits: L&T)

In most of the cases, active anchorages are installed at the superstructure, whereas dead anchorages are placed within the Link anchor boxes. Installation of dead anchorages shall be done prior to the installation of the anchor boxes within the pylon.

At deck side it is necessary to fix the anchorage temporarily to the tapped holes of the bearing plate, before it is held in place by stressed strands. If strands are not going to be installed shortly

after placing the anchorage, the anchorage surface has to be protected by covers to protect it from corrosion and damage

The injection opening of the ring nut (on its circumference) must be covered by duct tape to protect it from dirt and corrosion. All other threaded blind holes on the ring nut or dead anchorage front surface, which are used for connecting the caps later on, shall be protected by duct tape as well.

- **Corrosion Protection of Wedges**

  Wedges have to be treated with corrosion protection compound prior to using them for anchoring the strands at pylon and deck side **(Fig 5.19)**. The corrosion protection material is heated up within a melting pot and wedges that are placed into a sieve are immersed. After this, wedges are stored for drying.

- **Preparation of Cable Sheathing**

  The HDPE-sheathing supplied in straight single lengths is welded together to the required length with the help of a butt-welding machine. The length of the sheathing depends on the current temperature of the PE-tubes.

  A confined environment should be used for welding in order to protect the respective pipe sections from weathering and dirt during the welding process. This can be done using a welding container or a welding shed.

  After welding to its required length, the sheathing needs to be equipped with all necessary cable components and auxiliary tools.



Fig. 5.19 Example of Strand Secured by Strand Clamp (Credits: L&T)

5.5.2.2 Cable Assembly

Once having prepared the sheathing, a crane lifts the upper end towards the anchor box at the pylon, **Fig. 5.20,** the sheathing is attached to the tube at the front of the Link anchor box using eye bolts. The first strand can be inserted and anchored within the dead anchorage. At deck the lower end is held up by a fork lift or similar to prevent the sheathing from being scratched on the ground. By using the forklift, the sheathing is positioned in front of the deck recess pipe and the first strand's end is pushed through the recess pipe and anchorage hole using a PE hose for guidance.

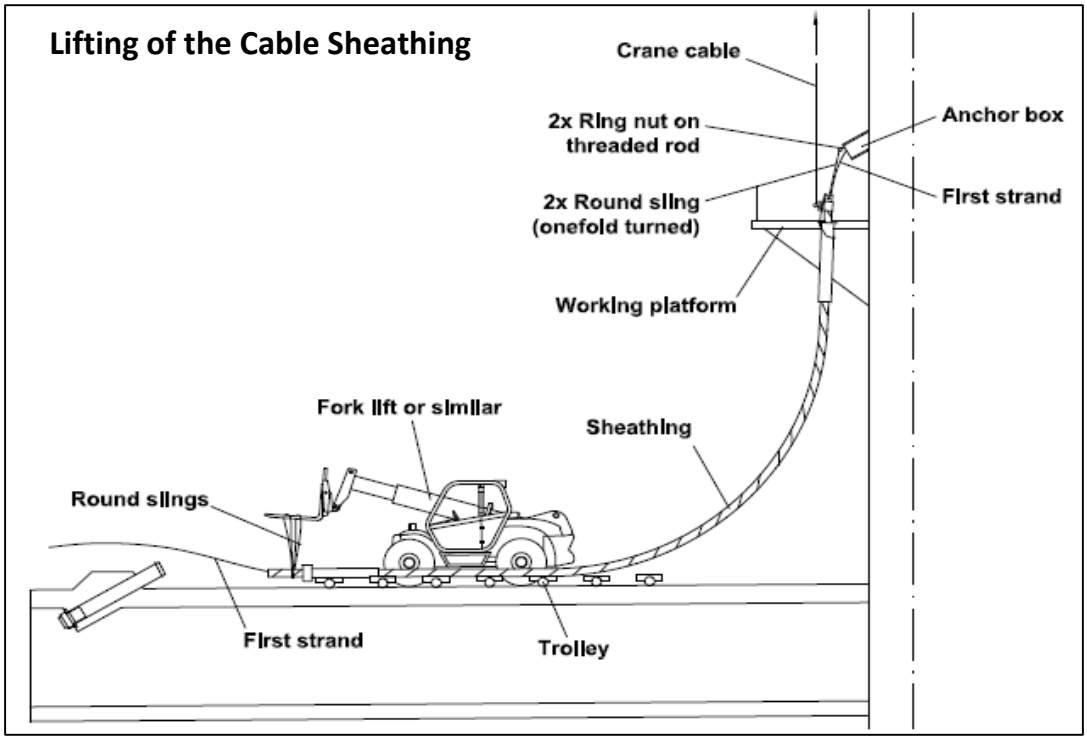


Fig. 5.20 Lifting of the Cable Sheathing

The strands are supplied reel-less and have to be placed in proper un-coilers. They will be cut to length prior to installation. Using a hydraulic pusher every strand will be pushed into the sheathing and be installed and stressed according to the **Fig. 5.21**.

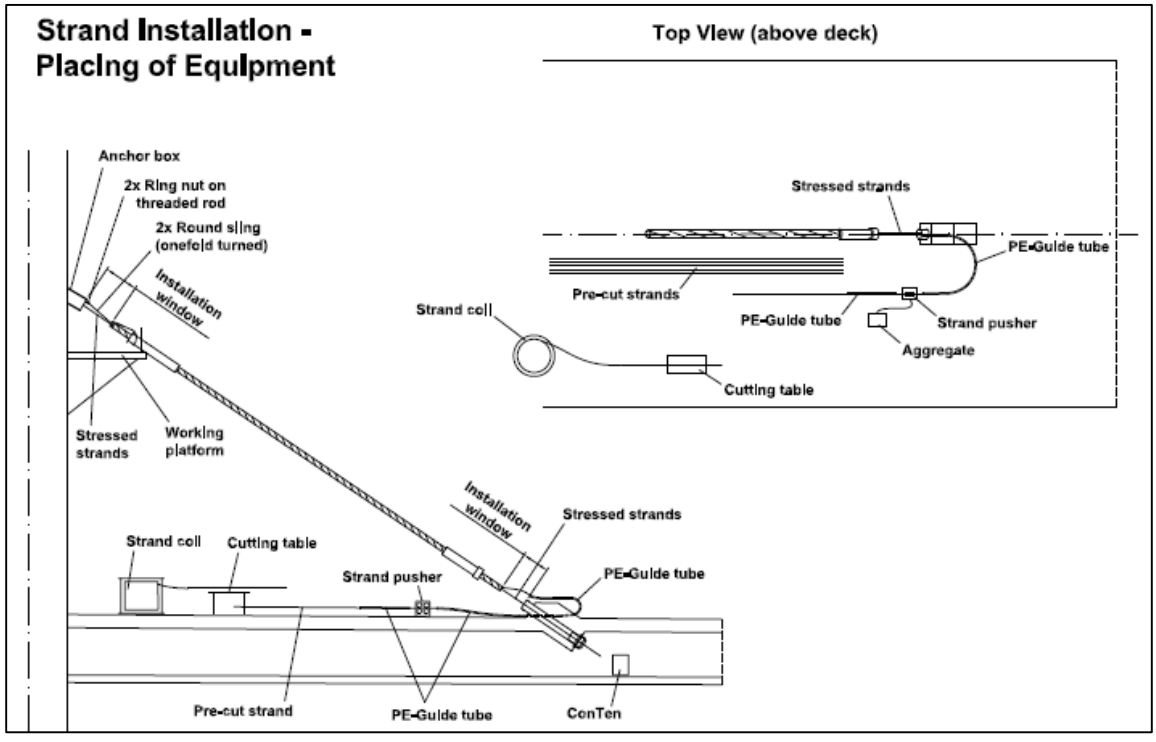


Fig. 5.21 Strand Installation – Placing of Equipment

Cable Lifting and Stressing of 1st Strand

The following operations are necessary:

i) The crane cable is attached to the pipe clamp and the sheathing is lifted towards the anchor box anchorage **Fig 5.22**. There the round slings with shackle are attached to the tip of the Link anchor box by eye bolts, a chain hoist can be used to adjust the position of the sheathing.



Fig. 5.22 Lifting of HDPE Pipe (Credits: L&T)

ii) By gently lowering the crane cable, the round slings take over the weight of the sheathing.
iii) At deck side, a fork lift or similar, supports the deck end of the sheathing and prevents it from being scratched on the ground and from kinks in the first strand caused by the weight of the sheathing.
iv) The deck end of the first strand is fed through the PE guide tube into the hydraulic pusher.
v) At top, the strand clamp is removed from the first strand, which is positioned in front of the anchor box.
vi) The pusher is used to move the strand upwards into the correct hole of the anchorage until it protrudes the anchorage by 70 mm.
vii) At this point a wedge is placed on the strand and fastened by applying three hammer strokes to a short pipe placed around the strand.
viii) At deck side the strand is removed from the pusher and inserted into the anchorage with the help of a PE-feeding hose that is pushed upwards through the corresponding cone hole.
ix) A wedge that is protected with corrosion protection compound has to be pushed onto the protruding strand together with a steel compression tube and must be placed in front of the cone hole.
x) The strand can then be stressed according to the stressing protocols
xi) After the strand has been stressed, the crane cable can be removed and installation and stressing of further strands may continue.

5.5.2.3 Stressing

The strands of the Stay Cable are stressed one by one. This will be done using any of the pre stressing system provider's patented system. Cable force may also be adjusted with the help of the gradient jack after installation of all strands.

The stressing records are the basis for all stressing operations. They are prepared in advance with all necessary data given by the structural engineer considering the deformation of the structure as

well as the forces, which have to be applied. In order to obtain a most accurate application of the stressing forces, it is recommended to carry out at least two stressing steps. In doing so, force differences in single strands due to differences in temperature during installation can be equalized.

- **Stressing of Strands**

  For stressing of Multistrand Stay Cables, two different methods are available;

  1. Single stressing system with mono-jacks
  2. Multistrand jack stressing (gradient jack)

  Appropriate stressing records for every single cable will be generated and submitted for approval with the data provided by the structural engineer in advance. Needed data are cable forces prior to and after stressing as well as the coordinates of center of the bearing plates (measured on the outer surface) before and after stressing. The procedure and principles of iso-tensioning is explained in separate section of stay cable systems.

- **Verification of Forces**

  The most important and common method for cable force verification is doing Lift-Off-tests with the mono-jack. Thereby a minimum of three strands of one cable will be lifted until the wedge is moving. With this actual pressure inside the jack, the force can be calculated.

5.5.2.4 Finishing works

***Installation of Dummy Strands**

Because the anchorages are not always fully occupied with the maximum number of strands, the free conical bore holes are filled with a short dummy strand and a wedge. Also within the steel bearing pipes at deck and pylon dummies have to be placed. Strand occupancy drawing is given by engineer in general, with detailed information on location, lengths and preparation of dummies.

- **Corrosion Protection Works at the Anchorages**

  At active anchors, the ring nut threads are injected as well as the short caps at dead end side. At stressing anchorages, the strand protrusions are covered by PE-protection tubes (filled with corrosion protection compound) is installed without being filled.

- **Activating the Sealing**

  The Threaded rods must be tightened applying a certain torque as per vendor's guidelines.

  The interspaces between conical boreholes, wedges, and strands are automatically filled by corrosion protection wax because the wax plate that has been inserted into the anchorages before will be compressed while activating the sealing unit.

- **Protection of Ring Nut Thread and Anchor Block Thread**

  Corrosion protection compound must be injected into the ring nut thread with a grease pump via a grease nipple at the outside of the ring nut until the compound exits at the thread run-out. After injection, a hexagonal socket plug with conical thread permanently replaces the grease nipple. The corrosion protection of the anchor block thread in front of the ring nut is the same as behind,

which was applied during anchor preassembly, and consists of a FILL material to be filled into the thread, and an FEU-tape to be wrapped around the thread afterwards.

- **Assembly of Caps at Stressing Side**

The PE-Protection tubes have to be cut to the required length according to the strand protrusions. Corrosion protection compound must be filled, which are then pushed over the strands with the filled end facing towards the anchorage, until they rest against the anchorage. The tubes have to be sealed with plugs at the rear side. Place the Cap with the Sealing ring on the ring nut and tighten the hex nut screws.

- **Assembly of Caps at Dead Anchorage Side**

The Foam lining has to be fixed to the inside of the Cap with adhesive glue. Remove the foam lining in front of the vent opening at the cap's back. Place the Cap on the anchor surface and tighten the hex nut screws, constantly and crosswise. The torque to be applied is as per system provider's specifications. From the bottom the cap is filled with corrosion protection compound until it exits at the vent opening on top. Take off all injection equipment and place plugs at both tapped holes

- **Installation of Steel Bearing Pipes**

After completion of installation and stressing, the strand bundles have to be compacted by a special compaction clamp to be able to install the steel bearing pipes. At deck, the steel bearing pipes have to have a distance as per vendor's specifications away from the recess pipe, while the respective distance at the anchor box measured from the end of the connection tube at the anchor box can be seen.

At deck side, installation of steel bearing pipes must not be done until all stressing works with jacks are finished completely. Compact strands by means of the strand compactor, which therefore is located between recess pipe and designated location of the steel bearing pipe. Position the two parted steel bearing pipe with PE linings inside fix the 4 screw fittings with following torques specified by the vendor. The strand compactor is removed.

5.5.2.5 Closing of the Sheathing

- **Temporarily**

Right after strand installation and also anytime the installation process is interrupted for not only a short period of time, the installation windows shall be closed / covered either by moving the sheathing and sleeve or by using tarpaulin. With this, ingression of large amounts of water can be avoided during construction phase.

- **Permanently**

After installation of all cable components, the sheathing is lowered onto the recess pipe. Slowly release the sheathing from its position by using a chain hoist in between round slings and the upper Link anchorage at the pylon. Lower the Connection part onto the O-Ring AV and onto the deck recess pipe flange. The sheathing is protected from being pushed upwards by 4 bolts M16 in the connection part. Fix the PE-sleeve's flange between the end of the bearing pipe of the anchor box and Flange Tube Anchor Box with Rubber Ring and the Flange Ring Sleeve, by using four bolts of M16.