GUIDELINES FOR DESIGN, CONSTRUCTION AND MAINTENANCE OF EXTRADOSED BRIDGES

(First Revision)

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Guidelines for Design, Construction and Maintenance of Extradosed Bridges

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Introduction

The Specialized Bridge Structures including Sea Link Committee (B-9) was constituted in 2021. The draft prepared by the Sub-Committee B-9.3 under the Chairmanship of Shri Alok Bhowmick was discussed in numerous meetings of Sub-group B-9.3 and finally approved in its meeting held on ________________ . The B-9 Committee in its meeting 19th September, 2022 after due deliberation approved the document for placing before the Apex Committee i.e. BSS Committee. The list of Personnel of B-9 Committee is as under:

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The revised draft 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 modified document was approved by the IRC Council in its Meeting held on 8th October, 2022 at Lucknow (U.P.) for printing.

SECTION 1

1.0 SCOPE

The guideline covers analysis, design, construction, testing, monitoring and maintenance aspects of extradosed bridges. It covers the following types of Extradosed Bridges:

a) Highway / Pedestrian Bridges with cast-in-place concrete deck and pylon

b) Highway / Pedestrian Bridges with precast segmental concrete deck and in-situ concrete pylon

The scope is restricted to concrete bridges only as extradosed bridges with composite deck and steel deck are rarely constructed. This guideline is applicable for a maximum main span length of 200m. Though this guideline is applicable to pedestrian bridges, its provisions should not be considered as fully covering the pedestrian bridges.

CHAPTER - 2

INTRODUCTION

2.1 General

2.1.1 *History of Extradosed Bridges*

The intrados is defined as the inner curve or soffit of an arch or in the case of cantilever constructed bridge, the soffit of the girder. Similarly the extra dos is defined as the upper most surface of the arch The term "extradosed was coined by" by Jacques Mathivat [1]. His proposal developed for the Arrêt-Darré viaduct with precast box girder sections, was the initiation point (**Fig.** 2.1). Mathivat's proposal substituted the internal tendons in the top flange of a box girder to external cables above the running surface, deviated over the piers by stub columns and anchored inside the box girder, which he called 'extradosed cables'. The extradosed prestressing applied at the uppermost surface of the voided box girder resulted in material savings of 30% when compared with a conventionally constructed cantilever Box Girder Bridge. Unfortunately, the proposal was not selected for construction instead a conventional cantilever post-tensioned structure was chosen. However the credit for the name 'Extradosed Bridge' for this concept is given to him. It should however be noted that in 1926 Tempul Aqueduct by Eduardo Torroja, one of the first examples of a modern cable-stayed bridge, with its shallow cables and heavy deck, which also qualifies for an extradosed bridge.

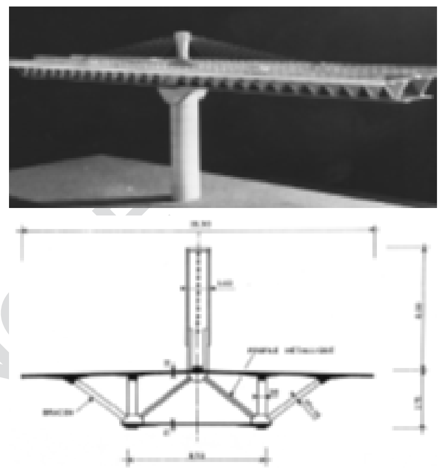


Fig. 2.1 :Arret-Darre Viaduct (Mathivat)

The Odawara Blueway Bridge, Japan is the first bridge in the world where Mathivat's concept was applied. Akio Kasuga, the Chief Bridge Engineer at Sumitomo Mitsui Construction Co. Ltd., was the designer for the bridge. The construction of this bridge was completed in 1994 (**Fig.** 2.2).

Over the past 10 years, extradosed bridges have become an attractive structural type around the world, due to the good results obtained with the first set of bridges constructed in Japan. This new type of construction, generally recognized as an intermediate solution between the cable stayed bridge and cantilever constructed prestressed box-girder bridge, as this construction uses the design and construction methods of the other two types. Hence this has become an interesting option. Despite such a large scale use however, there is not much of a widespread understanding of what this bridge type is, what the general design issues are, or what the general application of the bridge type may be. While there are many articles available on the

Fig. 2.2 :Odawara Blueway Bridge (Japan)

design of specific extradosed bridges, very little has been published on their design from a general perspective, except for an IABSE Guideline [8].

2.1.2 *Definition of Extradosed Bridges*

An Extradosed Bridge can be considered as a hybrid concept in the region of transition between girder bridges and cable-stayed bridges. A large number of options are available to designers for configuring such a bridge type to suit specific constraints for a particular project. The large number of Extradosed Bridges built over the last two decades is testament to that. In a cable stayed bridge the loads (permanent as well as moving loads) are carried by the stay cable. In a girder bridge loads are carried by shear and flexure of the girder and internal or external post-tensioned cables which produce permanent and constant stresses that act opposite to those produced by self-weight and moving loads. With a stiff deck and shallow cables, an extradosed girder behaves more like a prestressed concrete box girder although it has similarity in look with cables stayed bridges. Extradosed Bridge stays have a small inclination with respect to the roadway and, therefore, provide less vertical stiffness to the deck compared with a cable-stayed bridge. The shallowness of the extradosed cables directly carry only a small portion of the moving load. This is the basic behaviour of an extradosed bridge.

2.1.3 *Comparison of Extradosed Bridges with Cable Stayed Bridges and Girder Bridges*

Long span bridges fall into three categories namely girder bridge (constructed By cantilever method), extradosed bridge and cable stayed bridge. Segmentally constructed cantilever bridges are suitable for a span range of 70m to 130m. However there are a few exceptions with extraordinary lengths. Some of them are as follows :

a) Shivanpo Yangtze Bridge from Chongqing, China (Span 330m)

b) Stolma Bridge from Arland, Norway (Span 301m)

c) Rafsundet Bridge from Nofoten, Norway (Span 298m)

d) Sundoy Bridge from Leirfjord, Norway (Span 298m)

Cantilever bridges are generally constructed using precast segments or cast-in –situ. The depth of section at the support will be 1/16 to 1/20 th of span and depth at centre may be 1/30 to 1/50 th of span. The prestressing cables may be of internally bonded or externally mounted or a combination of both.

Extradosed bridges are generally constructed using segment technology, where some of the segments are supported by shallow stay cables which are anchored to the short pylon. Extradosed bridges are suitable for span ranges between 100m to 250m. The height of pylon will generally lie between L/8 and L/12 and depth of superstructure will lie generally between L/30 and L/50, where L is the main span length. The shallow pylon construction leads to shallower cable inclination. Shallow cables are often anchored closely spaced in groups which subdivide the main span length L into sections of 0.2L. Carrying the permanent loads of the bridge with such inclined cables lead to high cable forces. At the anchorages, the horizontal components of these cable forces impart prestressing force to the deck thus contributing to the post-tensioning of the girder. In extradosed bridges, the live load stress range in stay cables remains low.

The cable stayed bridges are adopted for a span range above 200m and upto 1100m bridge (Maximum span length for Russky bridge, Russia is 1104m). In a cable stayed bridge the height of pylon lies in the range of L/4 to L/5 and the superstructure depth lies in the range of L/50 to L/250. Due to larger height of pylon, the cables will be having a steeper inclination which will lead to high vertical component and low horizontal component of cable force. The deck is practically suspended between the cables and has minimum bending effect. This leads to relatively lesser superstructure depth when compared with extradosed bridges. The cables will be subjected to a larger live load stress range in addition to large vertical component of cable force at the cable anchorages. These two effects lead to adoption of special type of anchorages with high fatigue resistance for cable stayed bridges. In **Table 1**, the three bridge types mentioned above and their typical proportions are shown for comparison.

2.2 Salient Structural Forms

2.2.1 *Span Arrangement, Proportions & Statical Schemes*

Most extradosed bridges have two pylons longitudinally (i.e. pylon located above two different piers) with a 3 span module (**Fig.** 2.3). They generally have symmetrical configuration. For a typical 3 span module, the end spans are optimally about 50 to 70 % the length of the main span. If the end spans are longer, the sagging bending moments in the end span will be too high. If they are shorter, there could be uplift at the location of end support

Table 1
Comparison of Main Features between 3 bridge forms
(Cantilever Girder, Extradosed and Cable Stayed)

S. No.	Parameter	Cantilever Girder Bridge	Extradosed Bridge	Cable – Stayed Bridge
1)	Span Ranges	70m -130m	100m – 250m	200m – 1100m
2)	Span to depth ratio - At Pier - At Span	16-20 30-50	30-35 30-50	50-250
3)	Side Span to Main Span Ratio	Cast on form work 0.8 As Cantilever 0.65	0.45 to 0.69 Average 0.6	Steel superstructure 0.33 Concrete superstructure 0.42
4)	Pylon height to span	-	1/8 to 1/12	1/4 to 1/5

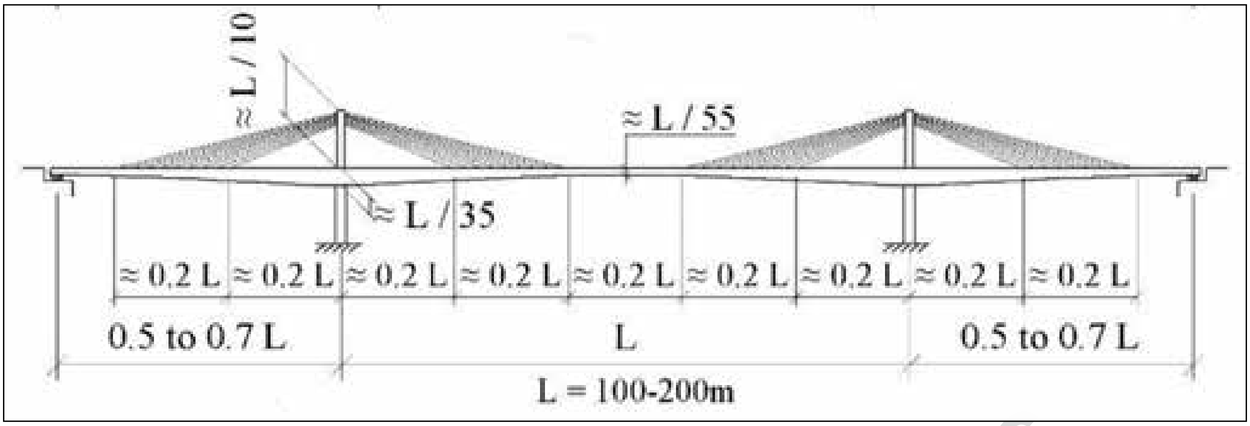


Fig. 2.3 Typical 3 span module Extradosed Bridge

However there can be many exception to this. Extradosed bridges with single pylon (i.e. pylon located above one pier), or else multiple pylons (i.e. pylon located above more than two piers) with symmetrical as well as asymmetrical configurations are possible and have been used in number of bridges.. Unlike multi-span cable stayed bridges, where design becomes a challenge as high deck bending moment is experienced when only one long span is loaded, the multiple span extradosed bridges do not face such issues, since the flexural rigidity of an extradosed bridge deck is much higher than the flexural rigidity of a cable-stayed bridge deck.

Extradosed bridges can more easily accommodate a curved deck layout in plan view. It is possible to design extradosed bridges with skew also, though it is very uncommon. Korong bridge in Hungary has been constructed with skew.

2.2.2 *Deck Cross Sections and Pylon Configurations*

Pylons for extradosed bridges can be either “lateral” or “central”. “Lateral” means that they are two pylons, located on either side of the deck. “Central” means that there is only one mast, located centrally in the middle of the deck. Both options have their own advantages and drawbacks. In most cases, the choice of pylon configuration is governed by the deck geometry.

For extradosed bridges, the configuration of the deck and the positions of the pylons transversely are important considerations. For wide decks, some countries (e.g. Germany) prefer to build two decks, in order to be able to replace easily without completely interrupting traffic. But from economic considerations (low capital cost) it is often preferred to build a single wide deck, with a central pylon or with lateral pylons or with both central and lateral pylons. **Fig.** 2.4 shows various cross-sectional arrangements of deck and **Fig.** 2.5 shows various pylon types used in extradosed bridges in India.

2.2.3 *Arrangement of Stay Cables and Cable Anchorages*

There is no specific international publication available for cable technology of extradosed bridges on their own. Mostly for extradosed bridges, the stay cable technology is used. In few cases, the stay cable technology have been used in combination with the external prestressing technology. Some international publications [4], [5] & [6] provide the technical information concerning the stay cables, addressing the concerns on durability, fatigue, dynamic response and other details. These publications however concern specifically stay cables technologies, irrespective of whether they are for cable stayed bridges or extradosed bridges.

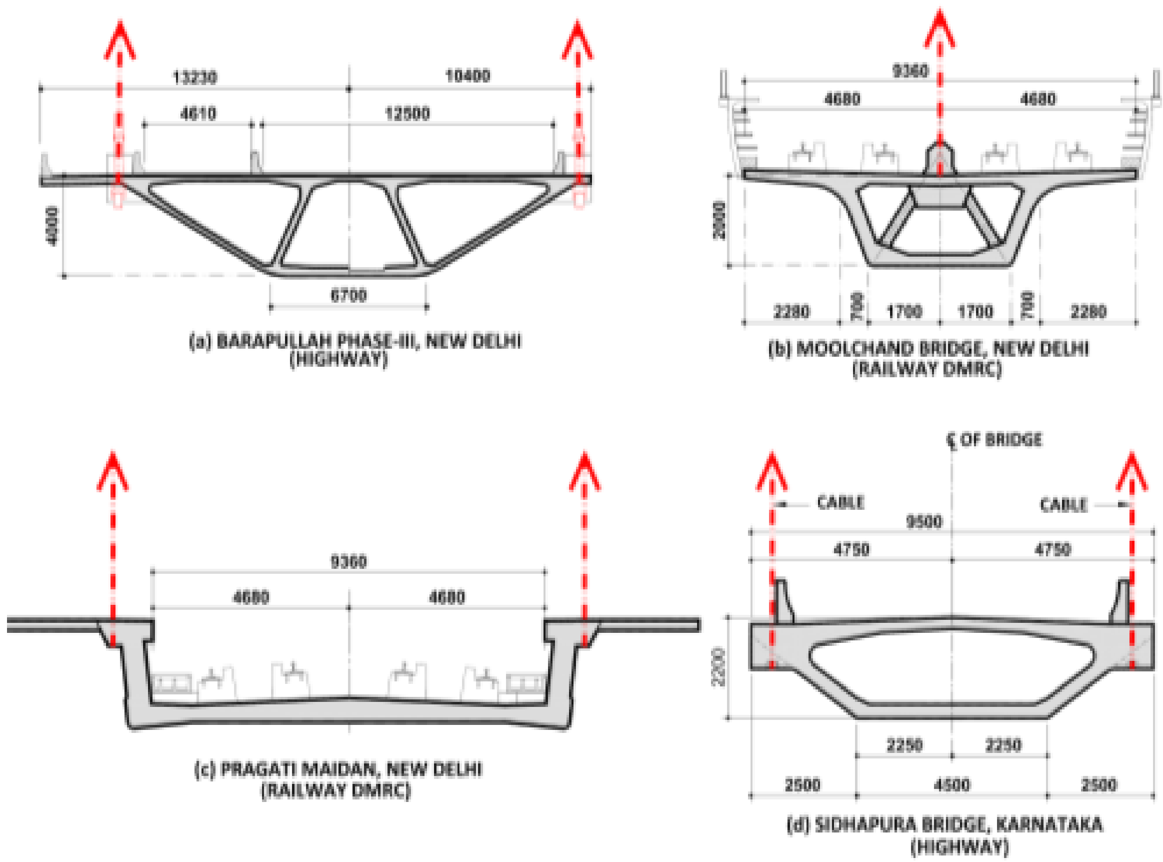
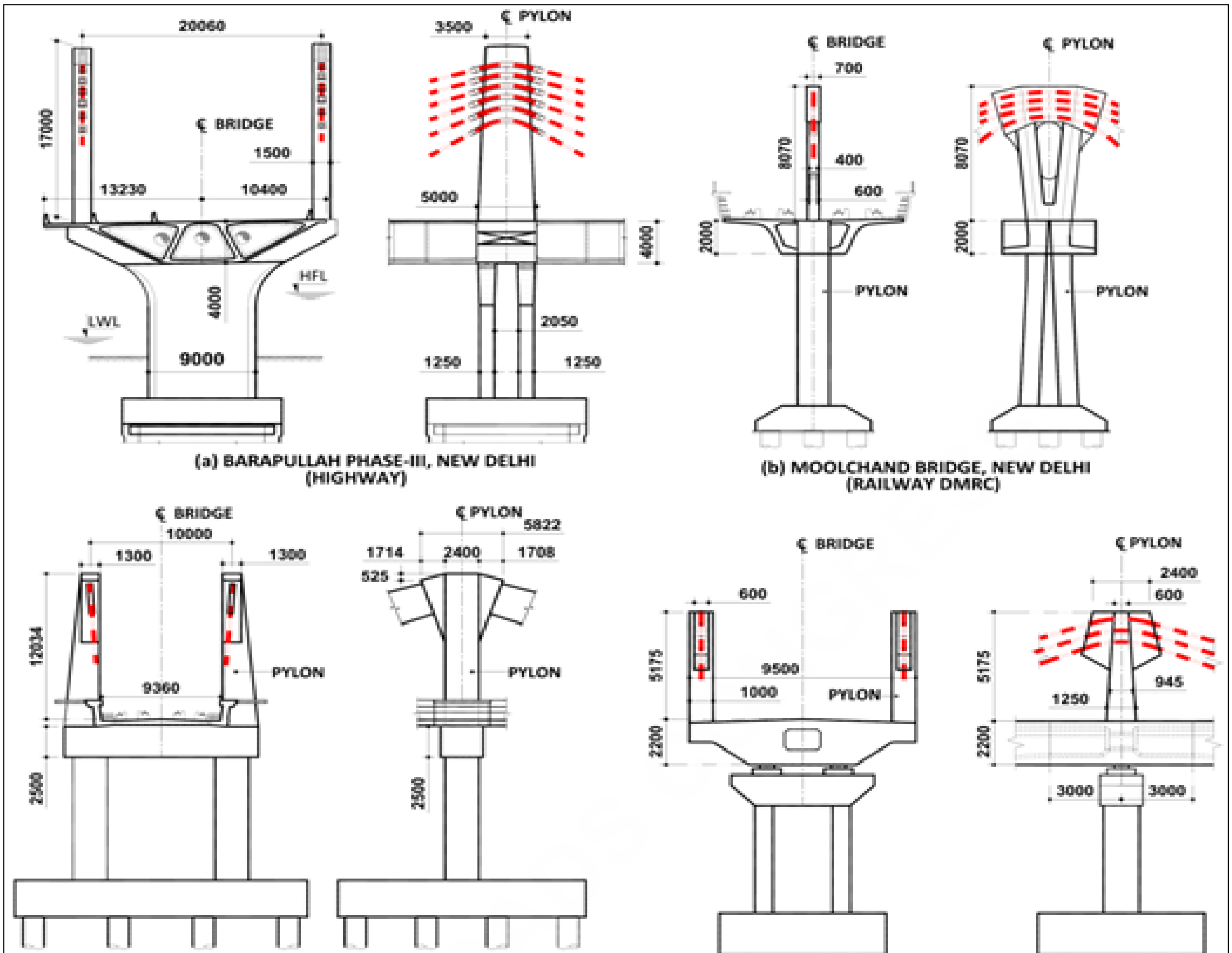


Fig. 2.4 Typical Deck Cross Section for Extradosed Bridges

Choosing the appropriate stay cable technology is an extremely important part of the design. The ultimate objective while choosing the stay cable system should be to provide a durable system for the design life of bridges.



(a) BARAPULLAH PHASE-III, NEW DELHI (HIGHWAY)

(b) MOOLCHAND BRIDGE, NEW DELHI (RAILWAY DMRC)

(c) PRAGATI MAIDAN, NEW DELHI (RAILWAY DMRC)

(d) SIDHAPURA BRIDGE, KARNATAKA (HIGHWAY)

Fig. 2.5 Typical Pylon Configurations forExtradosed Bridges

2.2.4 *Aesthetics for Extradosed Bridges*

These are long span bridges, where aesthetics plays an important role. Extradosed bridges with their short pylons with spread of stay cables harmoniously distributed along the deck can look very elegant.

Pylons must be carefully shaped in order not to look too bulky. Also, it should be noted, that the cable anchorages at the pylon are rather close to the users eyes and, therefore, well visible. The monolithic connections of deck to pylon or even to the piers below the deck are advantages to the extradosed bridge as less joints and bearings are provided. All these issues must be considered from the very start of the conceptual design.

2.3 Extradosed Bridge Practice in various parts of the World

2.3.1 *General*

From 1994 to 2010, over sixty extradosed bridges have been constructed worldwide, spread over in 25 countries with Japan taking a lead (30+ bridges) in design, construction, research, development and innovations [2]. Over the last 10 years, there is a steep increase in use of extradosed bridges around the world in a span range of about 100 to 200 metres. As of October 2022 more than 240 bridges have been constructed or are under construction around the world with China leading in terms of number of bridges built (82+) , Japan is in close second (With 64+ bridges) and India is third in rank with 21 bridges either constructed or under construction [3]. The rate of growth in India is so high that in a matter of few years, it may overtake Japan in this bridge type.

2.3.2 *Details of extradosed Bridges constructed in India (and under construction)*

Annexure-1 gives the list of Extradosed Bridges constructed / under-construction in India.

Fig. 2.6 Examples of Elegant Extradosed Bridges

(1) :Rittoh Bridge, Japan; (2) : St Croix River Bridge, Minnesota –Wisconsin, (3) : Bridge across the Daugava River in Riga, Latvia; (4) : WD-22 Viaduct over A4 Motorway, Poland)

2.4 Quality Assurance, Quality Control and Acceptance

Design and construction of extradosed bridge is a complex process. Design is very much construction driven and the interdependence between design and construction is much more than in case of routine run-of-the-mill bridges. Hence there is a need to implement total quality management (TQM) framework in the design and construction of these bridges. Good QC/QA program is a deliberate and systematic approach to reduce the risk of introducing errors and omissions into a design and construction. Provision of IRC:SP:112-2017 shall be strictly followed for ED Bridges. It is recommended to follow Q4 class of quality assurance for extradosed bridges.

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2. Steven L. Stroh, “Extradosed Prestressed Bridges” in *ASBI International Symposium on Future Technology for Concrete Segmental Bridges, held at San Francisco, 2008*
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CHAPTER - 3
DEFINITIONS

3.1. Definitions

1) Anchorage

A mechanical device, usually comprising several components (anchor head or wedge plate, bearing plate, socket, ring nut etc.), designed to retain the load in the stressed cable and to transmit the load to the cable-stayed / Extradosed structure. Anchorages can be as follows:

- Adjustable anchorage: Anchorage with a threaded nut or with shims, allowing an adjustment of the cable length without moving the prestressing steel relative to the anchorage.
- Fixed anchorage: Anchorage which does not allow adjustment of the cable length.

Anchorages may be further divided into:

- Stressing anchorages which permit stressing of the cable
- Passive anchorages which do not provide for stressing of the cable This should be in main body of relevant chapter

2) Anchorage Length

The length of cable within the hardware that serves to anchor the cable tension elements, including the length of the cable where the tensile elements are directed into the anchor hardware.

3) Actual ultimate tensile strength

Effective tensile strength of a main tensile element, defined as the mean of three ultimate tensile strength measurements on tests taken to fracture

4) Accessories

Auxiliary components such as anchorage caps, anti-vandalism pipe, sleeves, booths, etc.

5) Barrier / Corrosion Protection Barrier

Envelopment of the tensile element of the cable which protects the element or cable from environmental influences and their consequences, in particular corrosion. A barrier may provide protection by physical or electrochemical means, or through a combination of the two. Barriers can be of two types:

- External barrier: A barrier which is exposed to the outside environment.
- Internal barrier: A barrier which is directly applied to the tensile element

6) Bearing Plate

Element part of the structure which transfers the cable load at the anchorage interface.

7) Blocking Agent

A filler or coating that serves to prevent the passage of water to or around the MTE.

8) Cable

Complete system comprising one or several tensile elements (the MTEs), fitted with anchorages, including saddles, if applicable, and the relevant corrosion protection and accessories. Cable can be further distinguished into stay cable or extradosed cable according to application.

9) Cable Pipe

An enclosure encapsulating a bundle of tensile elements forming a cable in the free length.

10) Cable Specialist

If the cable supplier and the specialist contractor are the same company, this company is called the cable specialist.

11) Cable Supplier

The entity in charge of the design, qualification, and supply of the cable system.

12) Cross Tie

Element connecting the cables between each other and/or to the structure (bridge deck) to modify the natural period of vibration of the cable or to restrain cable displacements at particular locations.

13) CSIE

Cable-stay installation engineer (see Chapter 12)

14) Centralizer

A non-load-bearing device between the MTE bundle and the inside of the external stay pipe used to fix the position of the MTE bundle transversely within the stay pipe.

15) Deviator

The component which serves to deviate the path of the MTE at the end(s) of the transition length.

16) Deviator

A device (sometimes called tension ring or compaction clamp) which deviates the tensile elements radially to form a compact bundle in the free length.

17) Damping Device

A device to control cable vibrations.

18) Designer

The engineer responsible for the permanent work design and engineering during construction of the structure. The exact scope of work and role varies depending on the project.

19) Damper

Device fitted to a cable stay, using viscosity or friction to absorb and dissipate the kinetic energy imparted to it. Dampers may be attached to the cable stay and structure, or to the cable stay alone, in which case they work by inertia.

20) Duct

See definition of Stay pipe ?

21) Extradosed Bridge

An hybrid type bridge in transition between a girder bridge and a cable-stayed bridge, with a short tower and a relatively stiff superstructure.

22) Extradosed Cable

A cable which is used in an extradosed bridge.

23) Effective Strength

Quantity, comparable to a stress, defined as the ratio of the actual ultimate tensile strength F_{AUTS} to the nominal sectional area S.

24) ESC (Environmental Stress Cracking)

Cracking due to environmentally induced stresses

25) Free Length

Part of a cable between transition zones of cable anchorage and/or saddle beyond the cable anchorage or saddle and transition zones.

26) Filler / Filling Material

An interface, filler, blocking agent, or coating preventing the penetration of external contaminants to, or migration along, the tensile element and within the anchorage.

27) Fatigue Life

Number of cycles of loading of given amplitude that the stay cable must withstand without rupture.

28) Fatigue Range

Difference between the maximum stress σ_{max} (or the maximum deviation angle σ_{max}) and the minimum stress σ_{min} (or the minimum deviation angle σ_{min}). The fatigue range is twice the amplitude of the fatigue cycle, defined as the difference between the extreme values and mean values.

29) Fatigue Strength

Range of normal $\Delta\sigma = \sigma_{min}$ a cable stay can withstand for 2 million cycles without rupture and, by convention, below which the fatigue life is infinite.

30) Fretting Corrosion

Wear induced by small relative movements.

31) Fretting Fatigue

Fatigue induced by small relative movements.

32) Grout

A mixture of Portland cement, water, and approved admixtures. (Refer to PTI M55.1, "Specification for Grouting Post-Tensioned Structures").

33) Guide pipe

A tubular element (sometimes called recess pipe) which provides passage for the cable through the structure to the bearing plate. It aligns the cable at either the deck or the tower, and against which the cable damper or alignment device acts to resist lateral motion of the cable.

34) Guiding Device

A device (sometimes called elastic bearings) located behind the anchorage which transfers transverse forces to the structure to protect the anchorage from bending stresses. Optionally the guiding device can be incorporated within the anchorage, or combined with the deviator.

35) Guide

Guiding device located a few metres from an anchorage, constituting a fixed point which totally or partially impedes lateral cable-stay movement.

36) Galfan®

Alloy of zinc and aluminium sometimes used instead of zinc as a protective metal coating (Golfan® coating)

37) Goodman-Smith Diagram

Diagram giving the range of normal stress $\Delta\sigma$ a stay cable can withstand for 2 million cycle, for the maximum tensile stress σ_{max} or mean tensile stress $\sigma_m = (\sigma_{max} + \sigma_{min})/2$.

38) Guaranteed Ultimate Tensile Strength Class f_{class}

Quantity, comparable to a stress, which indicates the tensile strength per unit area of a main tensile element, in accordance with a reference standard.

39) Guaranteed Ultimate Tensile Strength Class F_{GUTS}

For PWC and PSC stays, guaranteed tensile strength of the parallel strand or wire, defined as the product of the strength class f_{class} of the MTE and its nominal sectional area S.

40) HDPE

High-density polyethylene.

41) Jaw

See Wedges.

42) Lollipop

A length of single strand, bar, or wire encapsulated with a single barrier material or compound over its length.

43) Lifetime / Service Life / Design Life

The planned period of use of the structure, or parts of it, for its intended purpose with the anticipated maintenance. It must be specified by the owner.

44) MTE (Main Tensile Element)

Individual prestressing steel elements which transfer axial tension load from one anchorage to the other anchorage.

45) Manufactured Length of Strand

One continuous length of strand produced from a stress-relieving or low-relaxation treatment line.

46) Maintenance

A necessary periodic activity to ensure the achievement of the defined lifetime. It intends to either prevent or correct the effects of minor deterioration, degradation or mechanical wear of the structure or its components in order to keep their future functionality at the level anticipated by the designer / owner.

47) MLS

Multi-layer strand (see chapter 9).

48) Nested Barrier

A barrier in series with another barrier.

49) Nominal Sectional Area S

Cross-sectional area of a main tensile element defined:

a) Either as the sum of the cross-section of the wires of which it is made, in the case of multi-layer stands;

b) Or by weighing and dividing by a conventional specific mass, in the case of prestressing wires and strands.

50) Owner

Ultimate authority during tender, construction and operation of the structure.

51) Polyvinyl fluoride tape (PVF)

Tape with backing made from a glossy PVF film or equal (refer to Appendix C).

52) Prestressing Steel

Prestressing wire, strand, or bar as specified in Section 3.0 used as the MTE of the stay cable.

53) PSC

Parallel strand cable.

54) PWC

Parallel wire cable.

55) Saddle

A structural device that allows both deviation of the MTE and transfer of stay force into the bridge structure without breaking the continuity of the MTE. (These provisions do not cover frictionless saddles.

56) Sheathed Strand

Prestressing steel strand coated with a corrosion-inhibiting coating and covered by an extruded polyethylene or polypropylene material.

57) Stay pipe

The external covering of the assembled cable, comprised of either high-density polyethylene (HOPE) pipe or metal pipe, or other materials and shapes to provide part of the corrosion protective system, and/or to control temperature variation and / or rain-wind-induced vibrations.

58) Saddle

A support between a cable and the structure, transferring primarily deviation forces and secondarily differential axial forces.

59) Sheathing

A factory-extruded high-density polyethylene (HDPE) around each individual strand.

60) Specialist Contractor

The entity in charge of the installation of the cable system. This entity must be trained and qualified by the cable supplier whenever the system is not installed by the cable supplier.

61) Spacer

A non-load bearing device between or around the tensile elements within the free length to fix the position of the cable pipe interior relative to the tensile elements.

62) SCSC

Specialist cable-stay contractor (see chapter 12).

63) S-N Fatigue Curve (Wohler Curve)

Curve giving the lifetime (number of cycles, on logarithmic horizontal scale) versus the stress range (on logarithmic vertical scale), for a particular element or detail, for cycles of constant range.

64) Stabilizing Cable

Cable restraining cable stays transversally to prevent them vibrating in the wind.

65) Strain under maximum Load, A_{gt}

Strain in a main tensile element when the peak load is reached during an ultimate tensile strength test.

66) Strand

Generic term covering:

a) So-called 7-wire prestressing strands (six wires wound helically round a straight central wire);

b) Multi-layer stands (MLS), assemblies of wires wound helically round one or more core wires, in several successive layers. They layers can be wound either all in alternating directions (crossed) to reduce torque, or all in the same direction to avoid point contacts.

67) Stuffing Box

Device located at the end of the anchorage in some cable-stay systems, through which the main tensile elements of the stay cable are inserted, and serving to provide a watertight seal between the free length and the anchorage zone, to contain grout, for instance.

68) Swaged Sleeve

Cylindrical sleeve for anchoring a strand; it is placed over the end of the strand and deformed in a swaging (hammering) machine.

69) Transition Length

The length of cable where the MTE is deviated from its neutral grouping of individual prestressing materials within the stay pipe to the grouping of the individual prestressing materials at the anchorage.

70) Transition Zone

The length of the cable where the tensile elements are supported by guiding devices and/or deviated from their arrangement in the free length to their arrangement in the cable anchorage.

71) Wedges

Steel parts in groups of two, three or four constituting a conical jaw for anchoring any strand by wedging it in a tapered hole machined into an anchorage block.

72) Wire

Individual steel wire, the smallest component part of a stay cable, generally round (except for the specially profiled wires of locked-coil cables), with a high elastic limit obtained by cold drawing.

73) Upper Pylon : Portion of pylon above deck level.

74) Lower Pylon : Portion of pylon below deck level.

75) Central Pylon : The middle pylon where there are three pylons in the transverse direction.

76) Lateral Pylon : Outer most pylons bounding the carriageway in between.

CHAPTER - 4
PLANNING CONSIDERATIONS

4.0 General

Extradosed bridges are a hybrid type in transition between a girder bridge and cable-stayed bridge with a short tower and a relatively stiff superstructure [7]. Thus, an extradosed bridge imbibes the 'stiff superstructure' characteristic of cantilever girder bridges and the 'tower and cables' characteristic of cable stay bridges in some proportions. Therefore, planning aspects of an extradosed bridge in terms of layout, static scheme, deck & pier types, towers and cables configurations derive their inspirations both from cantilever girder and cable-stayed bridges.

4.1 Investigations and studies

A number of studies are required to be carried out for successful planning, design, construction, and maintenance of extradosed bridges. These studies have to be commensurate with the proposed site conditions, method of construction, and the magnitude of the bridge project.

Some of the studies like hydrological model studies, site specific seismic studies and wind tunnel studies, etc. are time consuming. These studies need to be conducted during the conceptual design or Detailed Project Report (DPR) consultancy which is used for formulating tender conditions and specifications.

4.1.1 *Economic viability study*

The optimum pylon and girder height should be determined primarily by economic considerations, the optimum values can vary depending on the design conditions. In other words, cable-stayed, extradosed and externally prestressed cantilever bridges can be treated as the same category of the structure reinforced by stay cables. The difference among them is the extent of load distribution between cables and girders.

The optimum girder height for extradosed bridges is normally 1/30 to 1/50 of the main span length and the tower height ranges between 1/8 to 1/12 of the main span length. In particular, the pylon height is nearly half of the optimum value of cable-stayed bridges. The significance of extradosed bridges lies in their ability to resemble cable-stayed bridges with conventional cantilever bridge construction cross sections. The quantities of materials used (**Fig. 4.1a**) for extradosed bridges lie between the quantities of long-span girder bridges and cable-stayed bridges. In the figure, the relationships between maximum span length and average thickness of main girder concrete (concrete volume / deck area) and the span lengths for previously constructed extradosed bridges are plotted [1] The regression line gives the concrete quantities for both single planes of cables and twin planes of extradosed bridges. However, as can be seen from the figure, most of the twin-planed extradosed bridge quantities are above the regression line [3]. In the state-of-the-art report on extradosed bridges, it is stated that concrete quantities range between 0.4 and 1.4 m^3 of concrete per m^2 of deck area with an average value of 0.9 m^3/m^2.

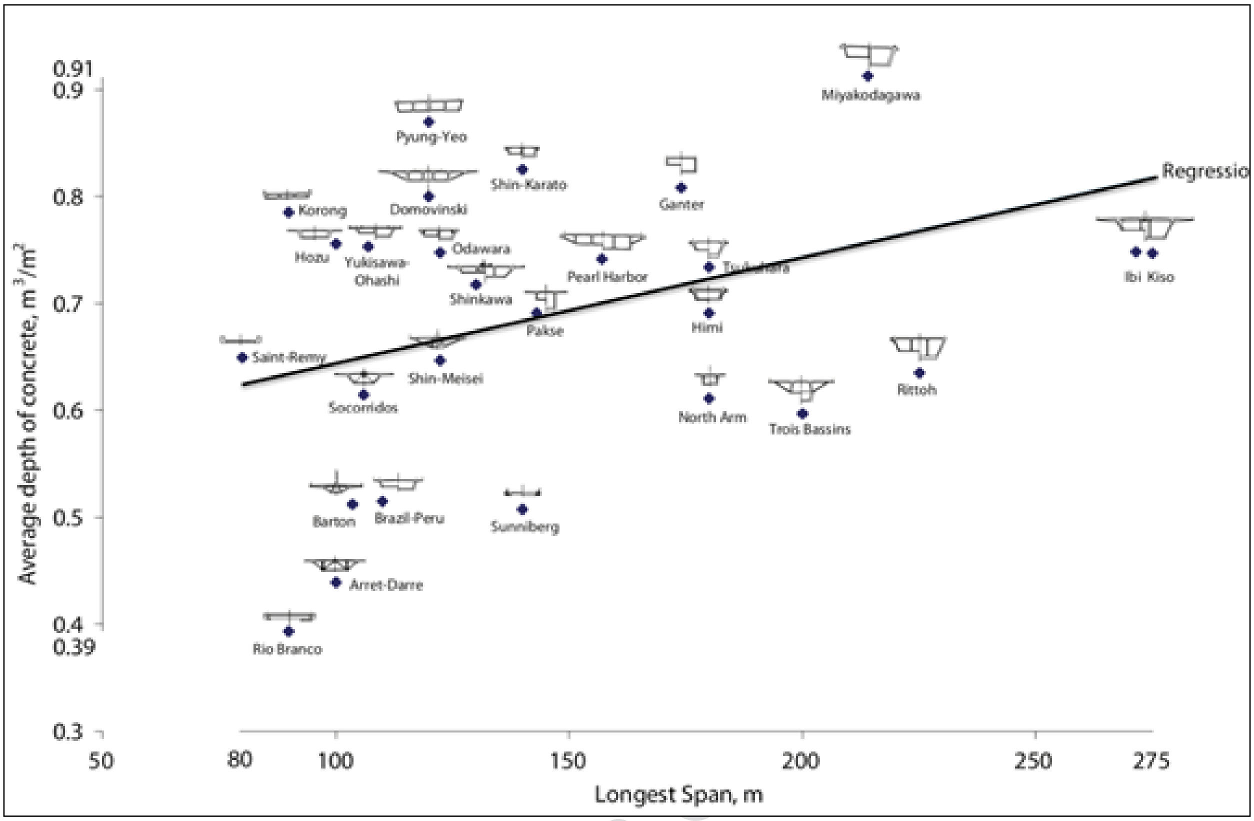


Fig. 4.1a : **Average concrete thickness** [1]

Extradosed bridges may be constructed similar to cantilever bridges, with no cable force adjustment of stay cables during construction and simultaneous stressing of stay cables and of cables within girders. Construction is also easier than for cable-stayed bridges, as the height of the main pylon is only around half that of a cable-stayed bridge.

The data collected for SED 17 on 54 Extradosed Bridges are plotted in **Fig. 4.1b**. The mean ratio of extradosed stays weight per unit bridge deck area is 16.7 kg/m^2, with a coefficient of variation of 55%. The amount of steel is larger in bridges with two planes of cables with an average weight ratio of 19 kg/m^2 compared with bridges with one central plane of cables with an average ratio of 13 kg/m^2.

In various specifications, the allowable stress in prestressing steel for extradosed cables is different from that of internal and external prestressing steel. Therefore, extradosed cables are distinguished from ordinary cables by some criteria. By using the concept of extradosed bridges, designers can select the distribution ratio of extradosed cables against the live load freely. This means that by selecting the load distribution ratio between stay cables and deck, one can expand the structural variety and choose the form with a larger degree of freedom.

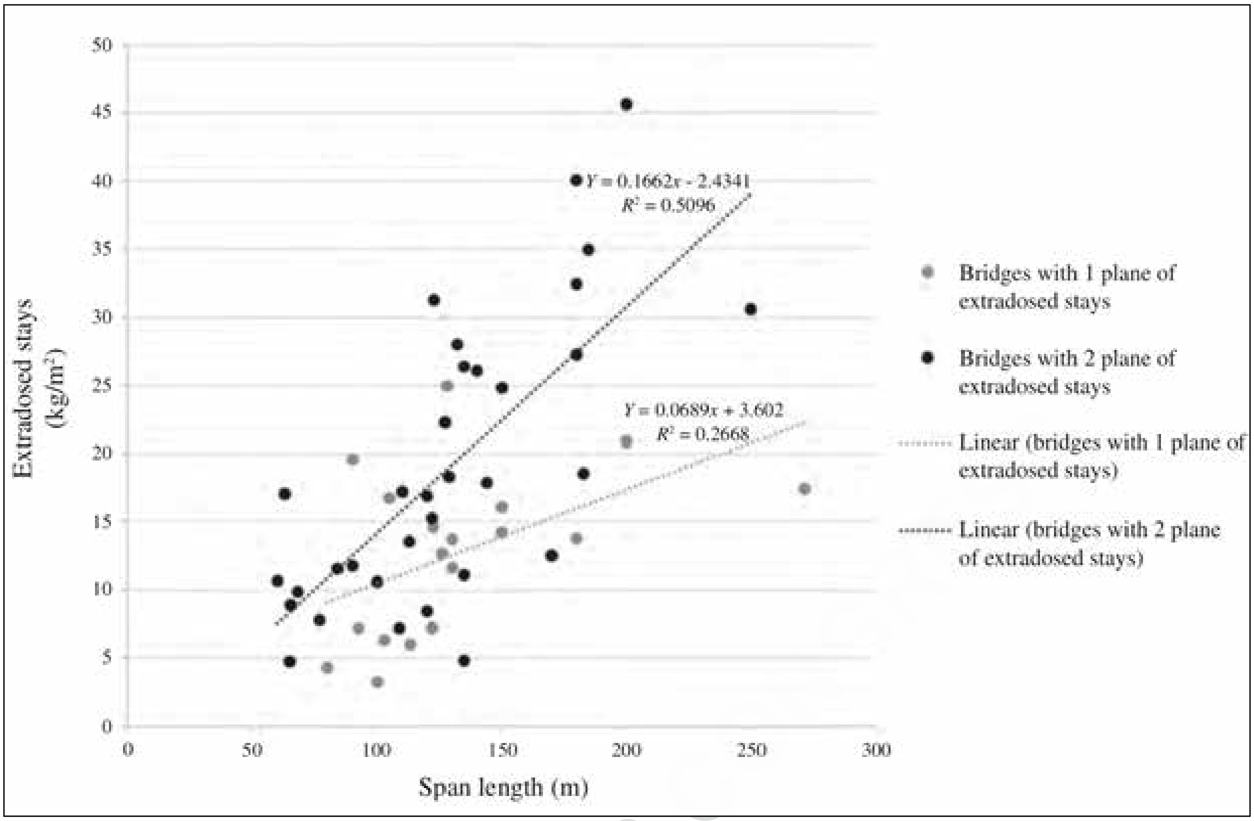


Fig. 4.1b : **Extradosed Cables Quantities** [3]

The maintenance requirement for extradosed bridges is higher when compared to cantilever bridges as cables are to be periodically inspected as such the same has to be given due importance during site selection and planning of structural forms.

4.1.2 *Topographical Study of the Construction Site and Surrounding Area*

While extradosed bridges are flexible in terms of various compositions using extradosed cables, pylons, and girders and can be more symbolic than normal cantilever bridges, it is important to make sure that the bridge design blends in with the surroundings. In addition, in the case of large extradosed bridges, it becomes necessary to carry out a study of the effects on plants and animals and the surrounding environment in general. Also, since the height from the ground or water surface to the top of the pylon may be quite high, it is necessary to check whether the pylons have any effect on air transportation or on communication. For wide meandering rivers, hydrological model studies may be required for deciding the linear waterway, bank protection, scour depth, etc. These site-specific studies should be conducted at DPR stage itself.

4.1.3 *Wind studies*

Cables with a total length shorter than 80 meters do not require any damping device [7]. When the maximum length of the cable is 80m, the span length works out to be around 200m. Thus, in case of extradosed bridges, where spans are in excess of 200m and the slenderness ratio of extradosed cable is large, it is necessary to consider the vibrating effects due to wind. Therefore, wind tunnel studies are not required for extradosed bridges up to the span of 200m. Since girders of “Long-span extradosed bridges” are more flexible than girder bridges and depending on site location, it is necessary to study stability against wind action. The geographical features of the construction site and a study of weather survey data (which should be continued during construction) are needed to study aero-dynamic stability for bridges beyond of 200m span or when the frequency is less than one second.

4.1.4 *Geotechnical, Geophysical, and seismic studies*

Guidelines on sub-surface exploration for bridges are covered in IRC:78. Geophysical surveys and investigations provide an effective tool to get preliminary information by covering a large area economically. They provide approximate parameters about the strata, which need to be confirmed by conducting the detailed geotechnical investigation later. Geophysical investigation methods, applicable to bridges are covered in IRC:123. For spans more than 150m in seismic zones IV and V, the I IRC: SP:114 specifies that site specific seismic studies are necessary. As this study is time consuming and also without which the contractor cannot do proper estimation during the tender design, it has to be carried out during the DPR stage itself. For long-span bridges either well foundations or pile foundations are preferred unless the underlying strata at shallow depth are rocky.

4.2 Overall Planning

Extradosed bridges have a high degree of design flexibility with regard to the various combinations of girder, cable, and pylon that can be used, leading to a high degree of freedom in the design process. In choosing design and form, it is imperative to fully understand the attributes of the structure and construction possibilities. On the basis of past experience during the preliminary design, the following aspects of planning need to be examined;

a) Span proportioning.

b) Types of girders.

c) Types of deck supports.

d) Pylon configurations.

e) Arrangement of cables.

f) Shapes of cable configuration.

g) Connections of pylon, piers and girders.

h) Choice of the appropriate type of foundation

i) Method of construction

Maintenance of extradosed bridges should be properly carried out based on the maintenance plan envisaged during the design stage. In maintenance planning, it is advised to establish maintenance policies (replacement and repair policies), inspection programs (inspection and frequency), and emergency policies for all structural members. These policies will require the installation of maintenance facilities such as scaffoldings and equipment to be provided in the cross-section and elevation. Moreover, in installing maintenance facilities, it is necessary to consider, the inspection and adjustment of lightning rods, obstacle lights, etc.

4.2.1 *Span proportioning*

Extradosed bridge spans lie in between spans of cantilever girder bridge and the longer span types like truss, arch, and cable-stayed. On the basis of past experience of the extradosed bridges constructed across the world, the span range preferred generally lies between 100 m and 200 m. However, a number of bridges have been constructed below and above this span range. In some cases, a hybrid design, with a steel section at the main spans also have been adopted to increase the span-range

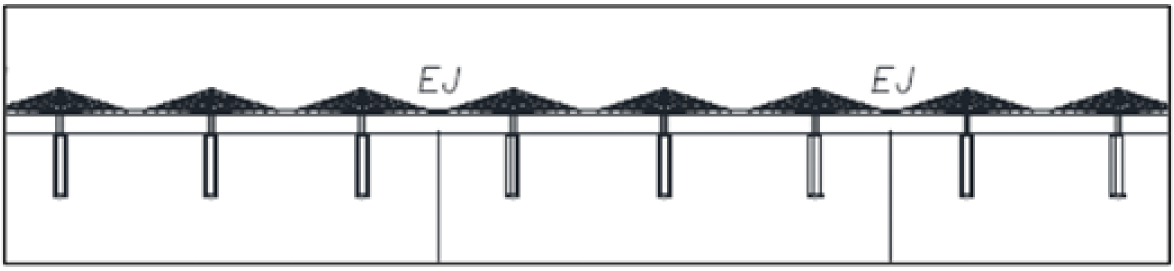
The ratio (L1/L) of span length between the main span (L) and side spans (L1) decides whether any anchoring/counterweight is required for the side span at the support location, moments demand in deck and pylon, and stress changes in the cables.

For extradosed bridges, this ratio falls somewhere between 0.42 of cable-stayed bridges and 0.65 of cantilever bridges. Based on the data available so far a reasonable side to main span ratio for an extradosed bridge is about 0.6. Sometimes this ratio meant for structural efficiency may not be adhered to, due to geometric and site constraints. One of the advantages of extradosed bridge is that this ratio can be accommodated depending upon site constraints without much impact.

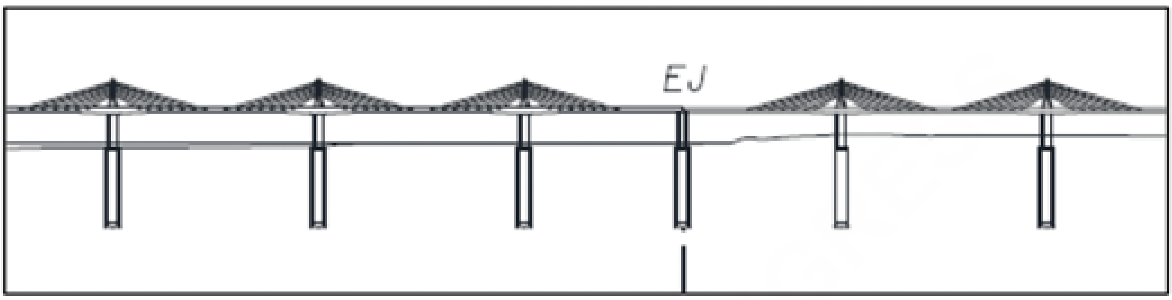
4.2.2 *Multi-span bridges*

For very wide rivers or waterways, several long spans are required for spanning the waterway. These river or creek beds usually warrant minimization of foundations culminating to long and multi-span bridges. In India long span prestressed concrete cantilever bridges used to be a preferred option for long and multi-span bridges on wide rivers like Brahmaputra and Ganga, etc. With the advent of extradosed bridges and their similarity in design and construction to cantilever bridges, this construction is becoming a preferred option lately over cantilever bridges.

Unlike in cable-stayed bridges, Extradosed bridges do not require backstay cables as the deck is very stiff akin to cantilever girder bridges. Thus extradosed bridges provide a viable alternative for long crossings with multi-span modules.



Provision of Expansion joint (EJ) at mid span bridge



Provision of Expansion joint (EJ) at pier location

Fig. 4.2 : Location of Expansion Joints in multi-span extradosed bridge[6]

In case of multi-span extradosed bridges, it is always preferable to avoid expansion joints anywhere between abutment locations. It is possible to avoid the expansion joints at mid span by provision of a pier support as shown at the bottom of the **Fig. 4.2**. The demerit of this system is that the spans are not uniform throughout as the adjacent spans to the pier supporting the expansion joint will be around 0.5 to 0.7 times the other regular spans. The various planning options of avoiding the expansion joints in between the piers are deliberated in the chapter 5 of this guideline.

However, if the bridge is too long the expansion joints, may also be provided in between the abutments. In long span cantilever bridges, the expansion joints used to be provided at midspan locations by providing suspended spans through articulation or by plunger type hinge bearings. Most of the bridges all over the world have witnessed excessive deflections at this location and failure of articulations and hinge bearings owing to wear and tear, pounding and long term creep over time.

The Plane Contact type Central Hinge Bearing (**Fig. 4.3a**) is said to be one of the options in overcoming the problems associated with older hinge bearings. This special Central Hinge Bearing consists of one male and one female part, which are anchored to the tips of the adjacent cantilever arms by means of high-tension PC bars. Special bearing devices are housed at the top and bottom of the male hinge, which slides over the stainless-steel plate provided in the female part. The special bearing devices are also featured with pre-compression applied during the assembling of the bearing in order to ensure perfect contact throughout its service life.

This special type of central hinge bearing works as a perfect and ideal hinge with additional freedom for translation in the longitudinal direction and also for rotation. The sliding load bearing flat interface (marked 1 in the left bottom of **Fig. 4.3a**) made of special wear resistant alloy with impregnated solid lubricant, will slide on stainless steel and offer minimum resistance and thereby chances of wear and tear can be eliminated and life enhanced. The elastomeric load bearing component (marked 2) not only allows rotation but also ensures absorption and even distribution of energy eliminating effect of continuous hammering action during passage of vehicle and thereby reducing chances of fatigue. Pre-compressed elements (marked 3) of the special bearing device and the special sliding elements is meant to maintain perfect contact between the male and female part and thereby maintaining deflection compatibility between the adjacent cantilevers. The specially designed pre-stressed anchorage system (marked 4) will maintain zero tension condition at the vertical contact surface of the girder and bearing and thus full capacity of concrete in compression would be utilized and fatigue in anchor bolts can be avoided. All the components of the bearing are easily accessible and replaceable.

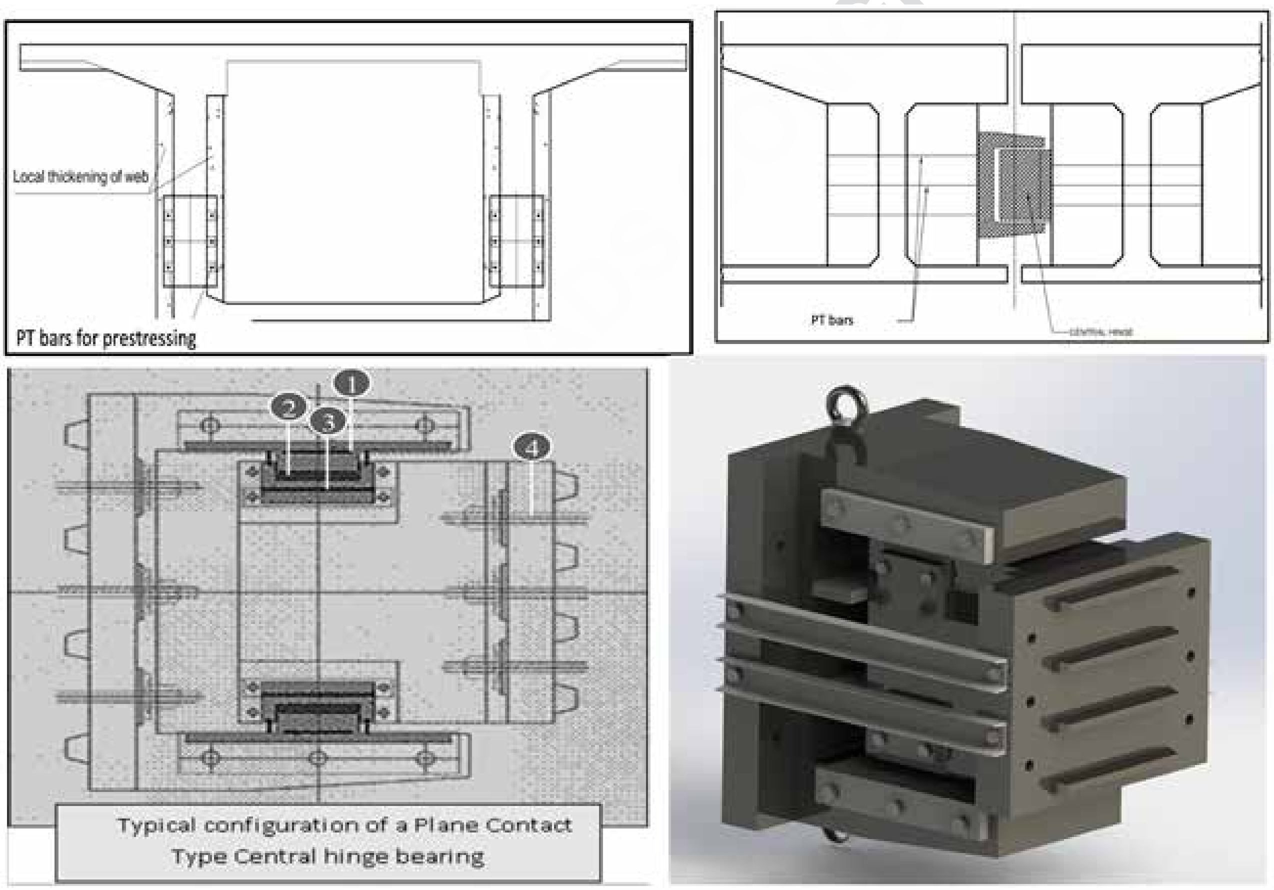


Fig. 4.3a : **Plane Contact CHB (Central Hinge Bearing) [2]**

Needle beams with bearings solutions are also adopted at mid-span for mating of the cantilevers. With this arrangement, it is possible to have uniform spans. Needle beam expansion joints (**Fig.4.3b**) are shear and moment-resisting structural steel systems having the combination of guided bearings at the top and bottom, allowing the expansion between two mating cantilevers. The structural form of the girders needs to be planned in such a way that inspection and maintenance of bearings and steel beams are possible in addition to the replacement of the same when called for. The merits and demerits of providing needle beam-type expansion joints are discussed in detail in chapter 5.

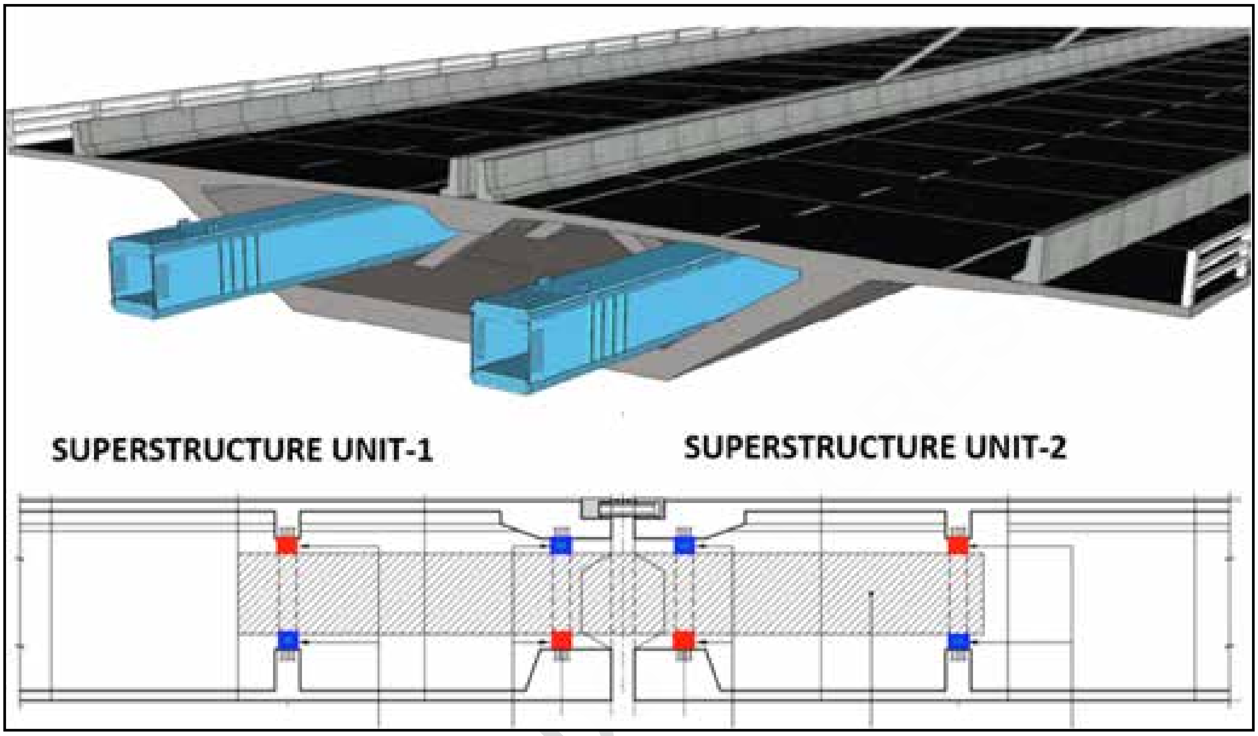


Fig. . 4.3b : **Needle beam option for Expansion Joints in multi-span extradosed bridge**

The extradosed bridge type can accommodate a modest curvature without special consideration of the structural system. The girder for extradosed bridges is typically a concrete box girder section, which can efficiently resist the torsional demands. The cables for extradosed bridges are typically only provided over a limited region of the span, and do not extend all the way to mid span of the main span or to the anchor piers in the side spans.

4.3 Girders for extradosed bridges

Span to depth ratio at the root as well as at the mid-span have been elaborated for extradosed bridges at various location of this document.

It is quite common to assume that depth to span ratio is same to all span ranges. However as per FHWA(1982) for extradosed bridges[4], when the data is plotted for the depth/span ratio as a function of span length, there seem to be a trend for increasing depth-to-span ratios for longer spans (**Fig. 4.4**).

In extradosed bridges, the designer can decide the structural proportioning by controlling the load distribution between the girder and cable system. This will give flexibility to the designer for adopting various options depending on site constraints and aesthetic requirements.

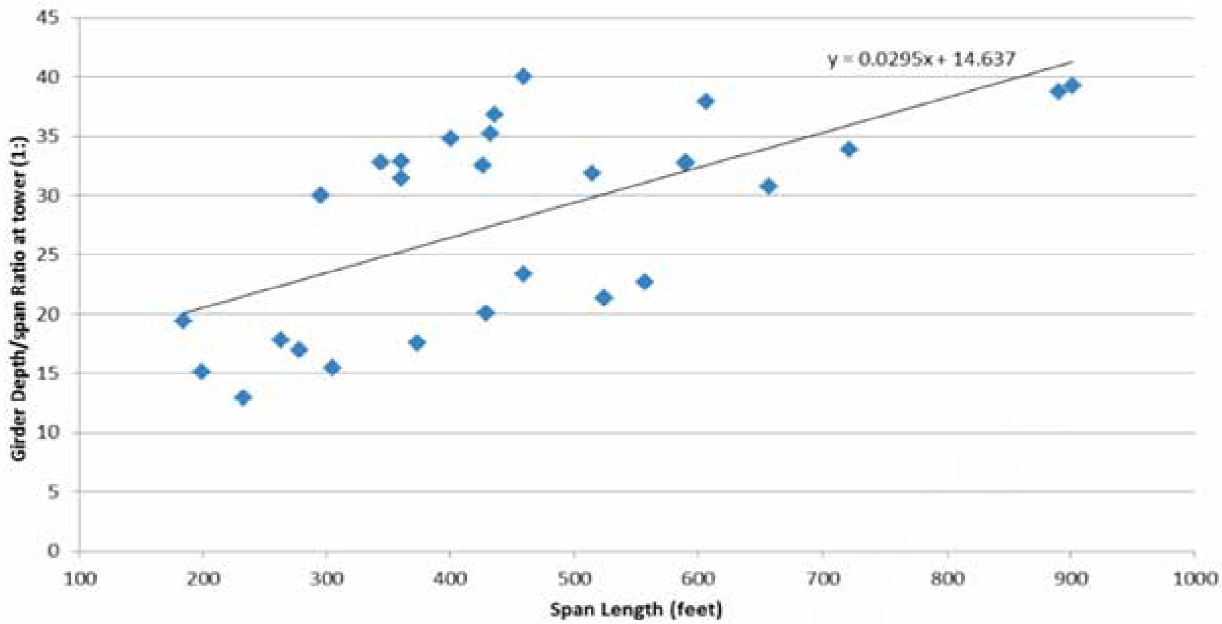


Fig. 4.4: Girder depth/span length comparison for full range of extradosed bridges [4]

Width of the extradosed bridges can vary depending upon the traffic lanes to be catered for. In case of narrow bridges, the prestressing forces are directly transferred to webs from extradosed cables. However, for wide decked bridges whether axially suspended or with multi-planar cables, the forces from the cables have to be transferred to webs via diaphragms, struts or cross-prestressing cables. This adds to the construction complexities in addition to cost.

Four major types of cross-sections and their characteristics are given in **Table 4.1** below.

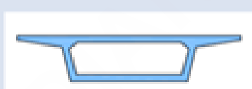
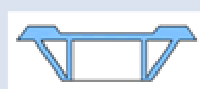
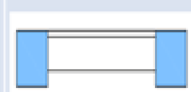
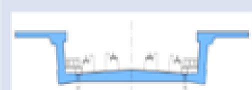
Types	Box girders	Wing girders	Edge girders	'U' trough
Cross section				
Attributes	✓ High torsional rigidity ✓ Lesser depth of girder ✓ Wider decks enabled ✓ Good wind resistant ✓ Utility accommodative & ease of maintenance	✓ Light weight ✓ Aerodynamic shape ✓ Suited for two planes of cables ✓ Suited for uniform depth of girders	✓ Light Weight ✓ Torsionally vulnerable ✓ Suited for two planes of cables ✓ Construction friendly	✓ Light Weight ✓ Torsionally vulnerable ✓ Suited for two planes of cables ✓ Adopted under special conditions ✓ Seldom adopted for Rail Over Metro bridges.

Table 4.1 Girder cross-sections and their attributes [6]

4.4 Piers and their Configuration

The piers transfer sectional forces from the girders and pylon to the foundations. Especially in extradosed bridges the pier head junction with girder and pylon has a complex connection with complicated stress conditions and the flow of forces. As such that area should be thought carefully and analyzed in detail. The shape and cross-section options (**Table 4.2**) of the piers depend upon the kind of connections between the pier, girder, and pylon and are also decided by intersecting traffic or river conditions in addition to aesthetic requirements.

The structural behavior of the bridge is influenced more by the design of the pier than that of the pylon. If the girder is simply supported on the bearings over the substructure, the bending in the piers is minimum. On the other hand, if it is monolithic with the piers, it will reduce the live load stress in the cables and allows for a more slender girder but increases the bending moments in the piers. When the girders are stiff, they can be made either monolithic with the piers or simply supported on the piers. However, stiff pylons require moment transfer between the pylon and the pier. The flexibility of the piers determines the extent of the bending moment in the pier due to temperature range, long-term shrinkage of the girder, and live load. Thus, to increase the flexibility, twin pier legs can be used to provide the desired rotational restraint.

For high earthquake conditions, twin wall or legs piers are preferable to increase the flexibility in longitudinal direction. Even in transverse direction, multiple column bent can be provided to lower the seismic force demand since the system is more ductile.

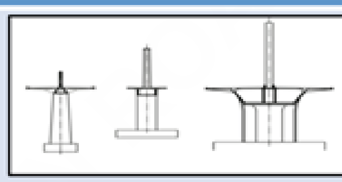
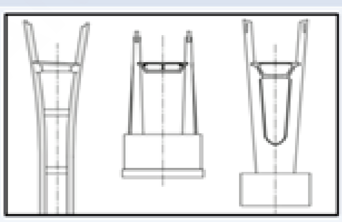
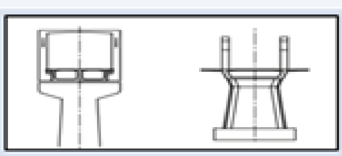
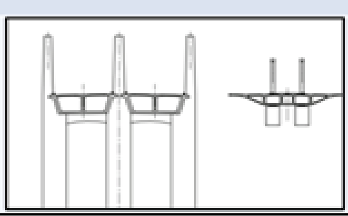
Types	Cross sections	Attributes
Wall type piers single/twin along the bridge axis with single pylon		✓ Most economical form for extradosed bridges ✓ If multiple span continuity is required, pier system may be made flexible by providing twin piers.
Twin pier legs across the bridge axis		✓ For bi-axial suspension with two pylon system, two pier legs are preferred. ✓ In seismic regions, twin pier legs may be preferable as they are flexible.
Wall type piers single/twin along the bridge axis with double pylon		✓ Pier system may be made flexible by providing twin piers. ✓ Deck can be embedded or supported on bearings integrated with pylons
Multiple pier legs with hybrid combination of pylons		✓ If there are three planes of cables supported by three pylons, the ideal solution is that of three piers. ✓ Multiple column bent results in lesser seismic forces due to higher ductile structural system.

Table 4.2 : Shapes and Types of piers [1]

4.5 Pylon, Girder, and Pier connection types

Types of girder support and connection of pylon, girder and pier and their attributes are shown in Table 4.3.

An integral rigid frame type of connection has a better economy and high degree of redundancy and does not require bearings at supports. However, continuous girder types with bearings at piers are also adopted due to conditions such as pier height and the number of spans.

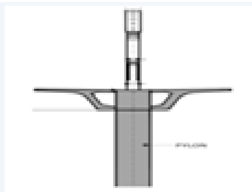
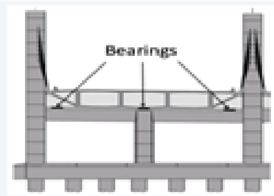
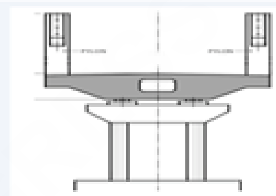
Types	Integral type	Continuous girder	
		Pylon & Pier integral	Pylon & Deck integral
Support Conditions			
Pylon, Pier & Deck connections	Pylon, pier & deck monolithic	Pylon, pier monolithic & deck on bearings	Pylon, deck integrated & on bearings
Attributes	✓ No bearings ✓ Large time dependent effects for multi-span ✓ During earthquake small deflection but large force at the integral junction of pier top & deck	✓ Reactions are transferrable to piers ✓ Small bearing sizes ✓ During construction, bearings are to be frozen ✓ Pier & Pylon attracts large earthquake forces	✓ Reactions are easily transferrable to piers ✓ Deck integration with pylon requires larger cross section ✓ Bearing sizes are larger ✓ Requires freezing of bearings during construction ✓ Small earthquake forces in the superstructure ✓ The natural period is smaller & first mode governs the earthquake design

Table 4.3 Comparison of girder support types and connection methods for pylon, pier and girder [6]

4.6 Pylon Shapes and Configurations

With the cable arrangement, the pylon front view (the view in the bridge axis direction) is an important element in the aesthetics of the landscape of the bridge. However, the determination of this pylon front view depends on the number of cable planes, cable arrangement, and girder width. The accepted and applicable pylon types and their attributes are shown in **Table 4.4**.

While deciding the cross-section of the pylon, in addition to ensuring durability and safety in limit states, due consideration must be given to cable anchorages and installation as well as space for tensioning and maintenance.

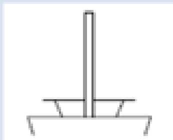
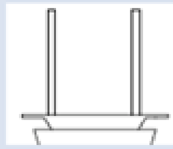
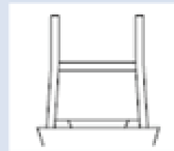
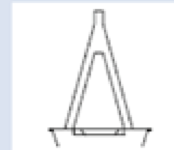
Types	Single Pylon	Double Pylons	Framed Pylons	A shaped-Pylon
Shapes				
Attributes	✓ Axial suspension at median ✓ Small out of plane stiffness for pylon ✓ Onlookers can have unobstructed view on one side while using	✓ Two planes of cables ✓ Deck suspended from the edges ✓ Small out of plane stiffness for pylons ✓ Suited for relatively small spans where pylon heights are small	✓ Two planes of cables ✓ Deck suspended from the edges ✓ Large out of plane stiffness ✓ Meant for large spans where pylons are taller	✓ Two inclined planes of cables ✓ Pylon out of plane stiffness is higher ✓ Requires larger pier width ✓ Pylon construction is cumbersome

Table 4.4 : **Pylon shapes and types and their attributes** [5].

The most important distinction of extradosed bridge from a cable-stayed bridge is pylon height. The pylon height dictates the design values, such as the stress variation in cables under live load (fatigue range), the cable inclination, and the load share between the deck and the cables, all of which are interrelated.

For extradosed bridges, the role of the cables is to act as external post-tensioning tendons and provide prestress to the deck. For an extradosed bridge, the post tensioning is elevated above the cross-section of the girder using a short pylon, and therefore provides a much larger eccentricity, hence more efficient use of the prestressing steel. However, if we continue raising the height of pylon, at some point, the vertical component of the cable reaches a force level that starts to significantly carry the vertical live load of the structure and the horizontal component imparting prestress to the deck reduces. This also means that the fatigue stress in the cable becomes more significant, and the bridge starts to behave more like a cable-stayed bridge, rather than an externally prestressed girder. Based on the population of existing and under construction extradosed bridges, the height (H) to span length (L) ratio is ranging between 1/8 to 1/12 (**Table 1**).

4.7 Cable anchorages at pylon end

Different methods for anchoring the cables to the pylon are shown in **Table 4.5**.

The spacing of anchorage and the edge distance of its components are determined according to the type of cable system, the cable system's place of manufacture (prefabrication or site fabrication), and material and shape of anchorage parts (concrete and steel parts, jigs, etc.).

Cable anchorage at the pylon and girder is the part where high local stress is expected to occur and attention should be given to reinforcement detailing as well as the arrangement, refer Chapter 10.

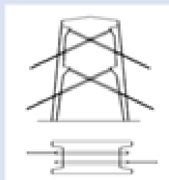
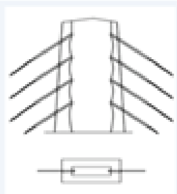
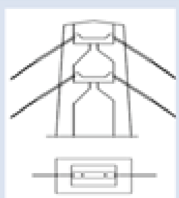
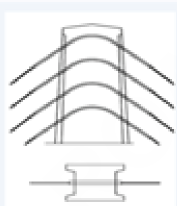
Method of Anchor	Types	Cross sections	Attributes
Anchorages are fixed separately	Cross		✓ Cables are crossing each other before anchored with in a closed cross sections ✓ Time tested for many cable stayed bridges ✓ Torsional checks are necessary
	Apart		✓ Cables are not crossing but anchored to shells ✓ Tension from anchored cables are resisted by steel plates or pre stressing tendons ✓ Spacing between cable anchorages can be small ✓ Easily inspect-able for maintenance.
	Coupled		✓ Cables are not crossing each other ✓ Steel link is used to counter the tension ✓ Anchorages are inspect-able for easy maintenance ✓ Larger cross sections are required
Cables are going through pylon continuously	Saddled		✓ Cables are continuous & passed through saddles ✓ Frictional force difference from one side to another needs to be accounted for ✓ Spacing between cables can be smaller ✓ Width restriction depending on bending radius

Table 4.4 Cross section shapes of Pylons for cable anchorages and their attributes [5]

When through fixing type is selected for cable anchorage structure of pylon, in some cases, it is restrained by the dimensions and cross-sectional form of pylon due to the provisions regarding minimum bending radius of stay cable (Chapter 10). Manufacturer of stay cables use different and patented systems for methods of anchorage.

4.8 Extradosed Cables

The cables play a very important role in distinguishing stay cables from that of extradosed cables. The basic role of the cables in a cable-stayed bridge is to develop elastic vertical reactions similar to beams on elastic foundations, for the purpose of suspending the deck, while in an extradosed bridge they are used to induce pre-stress into the girder.

In the side elevation of a bridge, there are three ways to arrange the cables as shown in **Table 4.6. Table 4.7** shows the characteristics of single plane and double plane cable arrangements. These should be planned with sufficient consideration of bridge's aesthetic design besides rational aspects of structural and construction conditions since the arrangement and number of planes of cables are closely related to the shape of pylon.

The cable system is a special structural member in extradosed bridges, its structure should be decided based on several factors such as superstructure's lane configuration and cross-sectional form, span arrangement, pylon structure, etc. When deciding the structure of the cable system, it is necessary to consider the relationships of features given below:

a) Strength characteristics and durability (anti-corrosion performance, vibration countermeasures) that are required in the design

b) Conditions for cable installation

c) Prefabricated or site fabricated

d) Construction efficiency (handling, carrying)

e) Economic efficiency

f) Future Maintenance method especially cable replacement

g) Types of patented cable systems available in India.

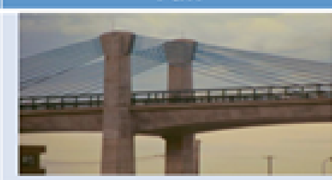
Types	Fan	Semi fan	Harp
Structure			
Attributes	✓ Cables are attached at the top of Pylon ✓ Complex cable anchorage structure at top ✓ Pylon completion before girder construction ✓ Large concentration of forces ✓ Pylon needs to be checked for buckling ✓ Since the angle of cables are steep, tend to behave more like stay cable	✓ Cables are attached along the length of the pylon ✓ In between Fan and Harp configuration ✓ As the angles of cables are shallower, acts more as an external cables in post tensioning ✓ Spacing between cables are relatively higher than fan type, structure of anchorages are lesser in complexity	✓ Cables are attached along the height and girder at equal intervals as such parallel ✓ Pylon and girder can be constructed simultaneously ✓ Less susceptible to vibration during earthquake ✓ There could be large changes in cable forces due to creep and shrinkage. ✓ Induces more pre stressing to the girder.

Table 4.6 : **The patterns of extradosed cable arrangement** [6]

In addition, the basic structure of the cable system should take into account vibrations induced by wind and secondary flexural stress due to the girder's deformation.

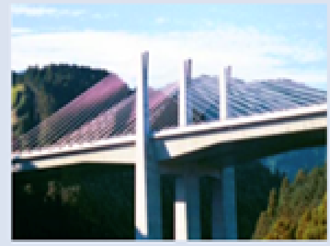
Cable Planes	Single plane	Double planes	Multiple planes
Cable Arrangement			
Attributes	✓ Width of the piers smaller ✓ Well suited for box girders from the torsional rigidity point of view ✓ The view from the bridge for the users are open and spacious	✓ Width of the piers larger ✓ Well suited for edge girder and wing girder configurations ✓ Torsional resistance of the girders are larger due to dual cable planes	✓ Normally adopted for very large width of carriage ways ✓ Normally substructure is multiple column bents ✓ Torsional resistance larger larger due to multiple planes

Table 4.6: **Single and double plane cable attributes** [6]

Regarding the arrangement of the cable system, the following aspects are required to be taken into account in order to satisfy the characteristics of the system.

a) Working space for prestressing cables should be ensured because prestressing in pylon is performed on high scaffoldings and prestressing in girder is conducted in confined space. If the replacement of cables is envisaged, working space during replacement should be planned. The anchorage components in the pylon are installed in a staggered arrangement in some cases.

b) In many cases, the girder is generally built by cantilever erection using cables. It is necessary to determine the arrangement of cables according to the erection method, i.e., whether cables should be anchored in every segment or every two to three segments.

c) The bending radius of cables through the fixing system pylon should be determined by confirming the safety of cables and the pylon's structural members (Chapter 8).

d) The safety of the bridge during cable replacement and the method of replacement erection should be examined and planned.

4.9 Choice of Foundation for Extradosed Bridges

Extradosed bridges come into the category of long-span bridges. Foundation structures for long-span bridges have some unique characteristics. These foundations are subjected to high vertical and lateral loads, requiring either large number of piles of larger diameter or large-sized well foundations. Choosing the appropriate type of foundation and appropriate span configuration would require following considerations :

a) Carrying out thorough geotechnical investigation

b) Choosing the appropriate type of foundation concept

c) Choosing appropriate design criteria and methods

d) Incorporating appropriate geotechnical model(s) in the analysis with due regard to use of soil springs and liquefaction depth where relevant.

The current IRC code on foundation design IRC:78, may not be fully geared to deal with all aspects of long span bridges. Specialist literature may be followed in such cases.

Reference

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7) Fib Bulletin 89, Acceptance of Cable Systems using prestressing steels, 2019.

CHAPTER - 5
PRELIMINARY DESIGN CONCEPT

5.0 The Concept of the Extradosed Bridge

5.1 Salient Features of the Structural Action

The most frequent configuration for Extradosed Bridges is a 3-span construction with pylons located on two piers. As brought out in Chapter 2, the side spans are approximately 50–70% of the main span length. If the side spans are longer, the bending moments will be excessive in the end span. If they are shorter, there could be an uplift at the end support.

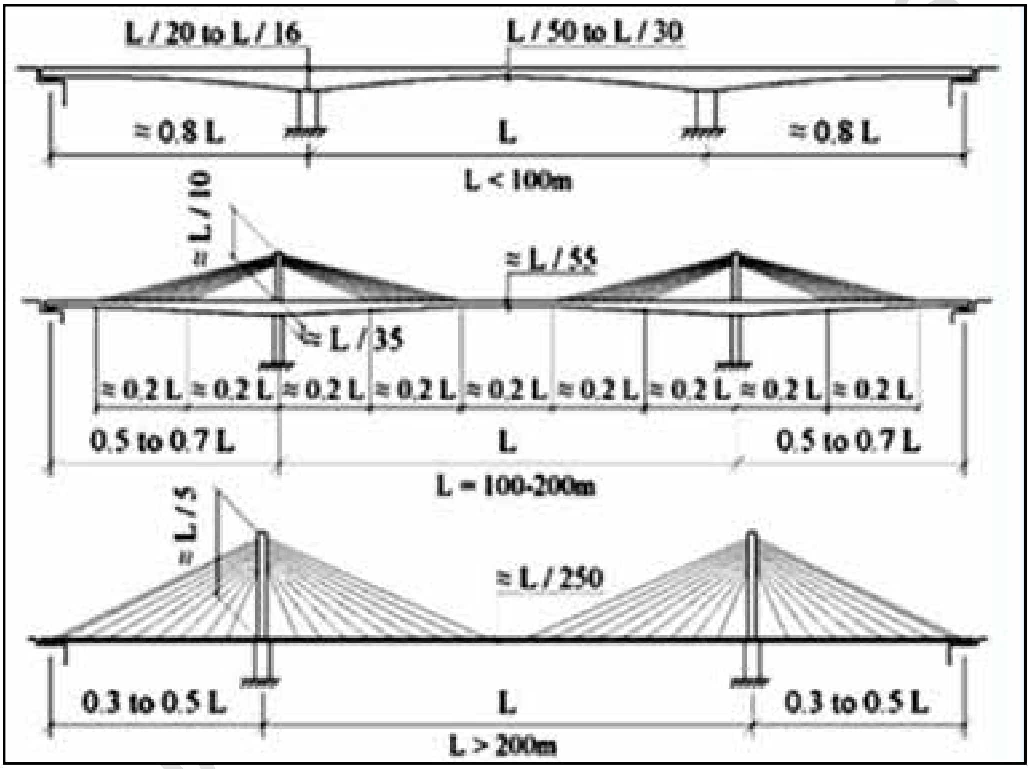


Fig 5.1 Characteristics of Free Cantilever, Extradosed and Cable Stayed Bridges [8]

Fig 5.1 gives the comparison of the typical layout of a variable depth cantilever box girder, an extradosed bridge, and a cable-stayed bridge. The short pylon height in the case of EDB results in a shallow angle of the stay cables with respect to the roadway, thereby providing very little vertical stiffness to the deck. On the other hand, the large horizontal components of the stay cable forces are instrumental in providing compression in the deck thus contributing to the post-tensioning of the Girder. The Extradosed Bridges usually require higher-strength concrete, because of the additional deck compression caused by the shallow angle of the stay cables. Most Extradosed Bridges are built in concrete, though there are a few examples in the world using a composite steel-concrete deck.

Since the stiffness of the stay cables is small, it is the deck that has to carry most of the loads in bending and shear to the supports. The decks of Extradosed Bridges, therefore, have to be made stiff and the prestressed concrete box girder section becomes an obvious choice in most cases.

The stay cables are progressively tensioned as the construction proceeds in cantilevering, and carry only part of the *dead load*, the rest being carried by the deck itself. The *live loads* applied after the completion of the bridge again are carried largely by the deck due to its enormous stiffness as compared to the stay cables which are hardly activated. In a typical extradosed bridge the stress changes due to live loads are in the neighborhood of 50 MPa [1]

5.2 The pier-deck-pylon connection

The variety of options available for the pier-deck-pylon connection have been discussed in Chapters 4 and 9. Making the piers monolithic with the deck is always a good solution as the bearings can be eliminated in this concept. Strains due to temperature, shrinkage, and creep do not become an issue if the piers are tall or can be made longitudinally flexible by resorting to the twin pier concept (Fig 5.2) or the substrata is not of stiff nature. The twin pier concept can reduce the moment of inertia to one-fourth as compared to a solid section as shown below:

Required Area of Pier = b*d

I_1=b(d)^3/12 …… Solid Section

I_2=2*b(d/2)^3/12= bd^3/48 …. Twin Pier

Mid-span jacking after the completion of the cantilevers and before casting the central stitch can also be usefully employed to reduce the effects of these strains on the substructure and foundations.

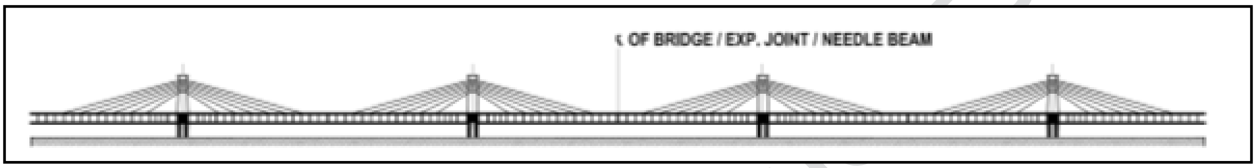
Fig 5.2 Reducing Pier Stiffness

The pylons are often made integral with the deck which helps in efficiently stabilizing them during construction and dependence on the pier table and stays for this purpose is reduced.

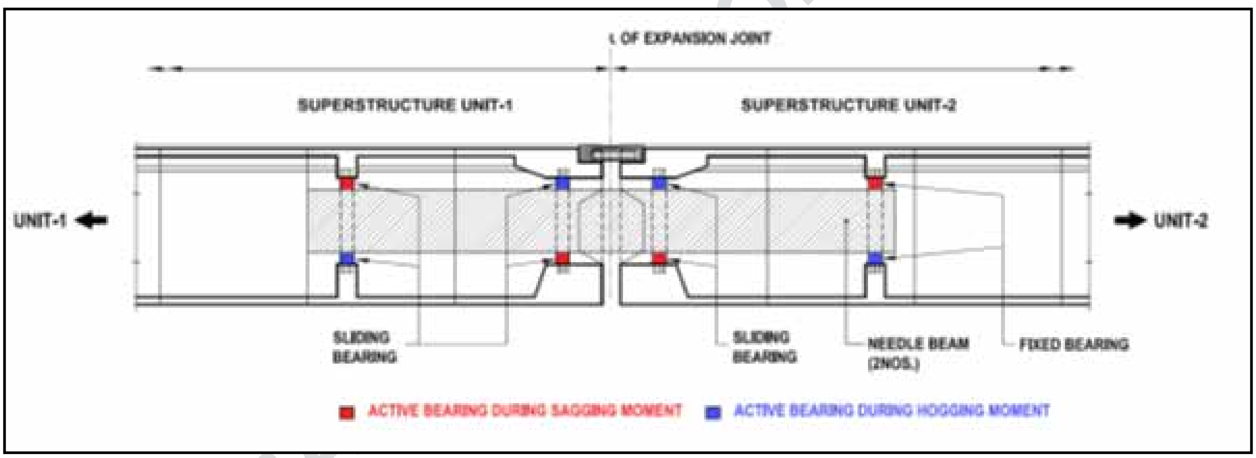
Some extradosed bridges are made fully integral, i.e. deck, piers and pylons are also monolithically connected.

5.3 Multi-span Extradosed Bridges (including Needle Beam)

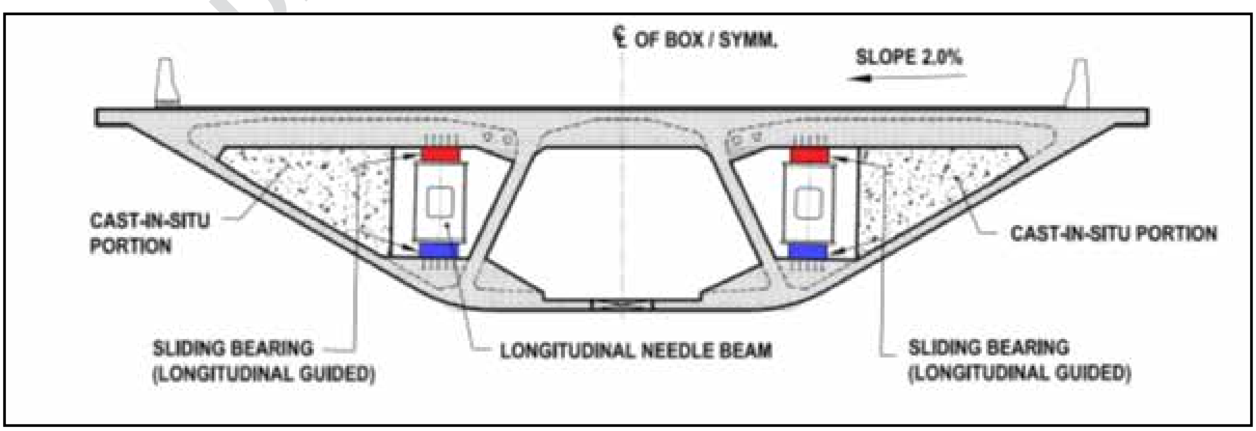
If the bridge is long as in the case of multi-span configurations some piers located far from the null points can be provided with bearings to allow movements. One of the options is to use expansion joints with “needle beams” or “plane contact (modern) central hinge” at midspans of Extradosed bridges with equal spans throughout the length (Fig 5.3a). Please see discussion in Chapter 4 in this regard and also other options discussed in para 5.4



a. Long Bridge with Expansion Joint Location at Midspan



b. Longitudinal Section



c. Cross Section

Fig 5.3 Expansion Joints at Midspan of Long Extradosed Bridges

The Needle Beam solution for the midspan Expansion Joint has been used in in India and abroad [2]. When Needle Beam solutions are provided, they require special attention, particularly to issues summarized below:

- Each Joint has 2 Steel Beams (say, 20t each) depending upon the number of cells in the box, 4 diaphragms, 16 bearings (Figs 5.3 b, c).
- The bearings have a design life much shorter than that of the bridge and will require to be replaced whenever required to ensure proper transfer of forces across the joint.-
- The Steel beam should be replaceable in case of corrosion, distortion/ damage due to accidental loading (fire, earthquake, etc.)
- The needle beam should satisfy all fatigue requirements.
- Adequate access and working space for inspection, maintenance and replacement should be provided.
- Bearings can lose contact with the parent structure during service (Fig 5.4) even if they were initially tight. The bearings should be so designed and provided such that it can be ensured that no loss of contact occurs for the entire design life of the bridge.

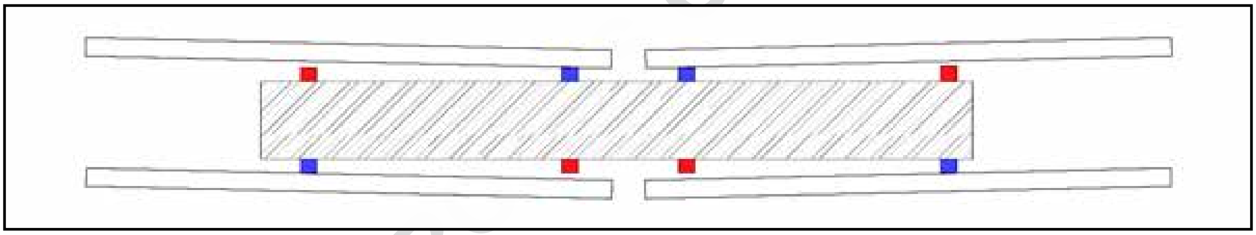


Fig 5.4. Midspan Expansion Joint: Bearings which were Initially Tight

5.4 Structural Arrangement Options

For long multi-span Extradosed Bridges some alternative structural configurations whereby the expansion joints at midspan can be eliminated and positioned at pier locations are discussed below and depicted in Fig 5.5.

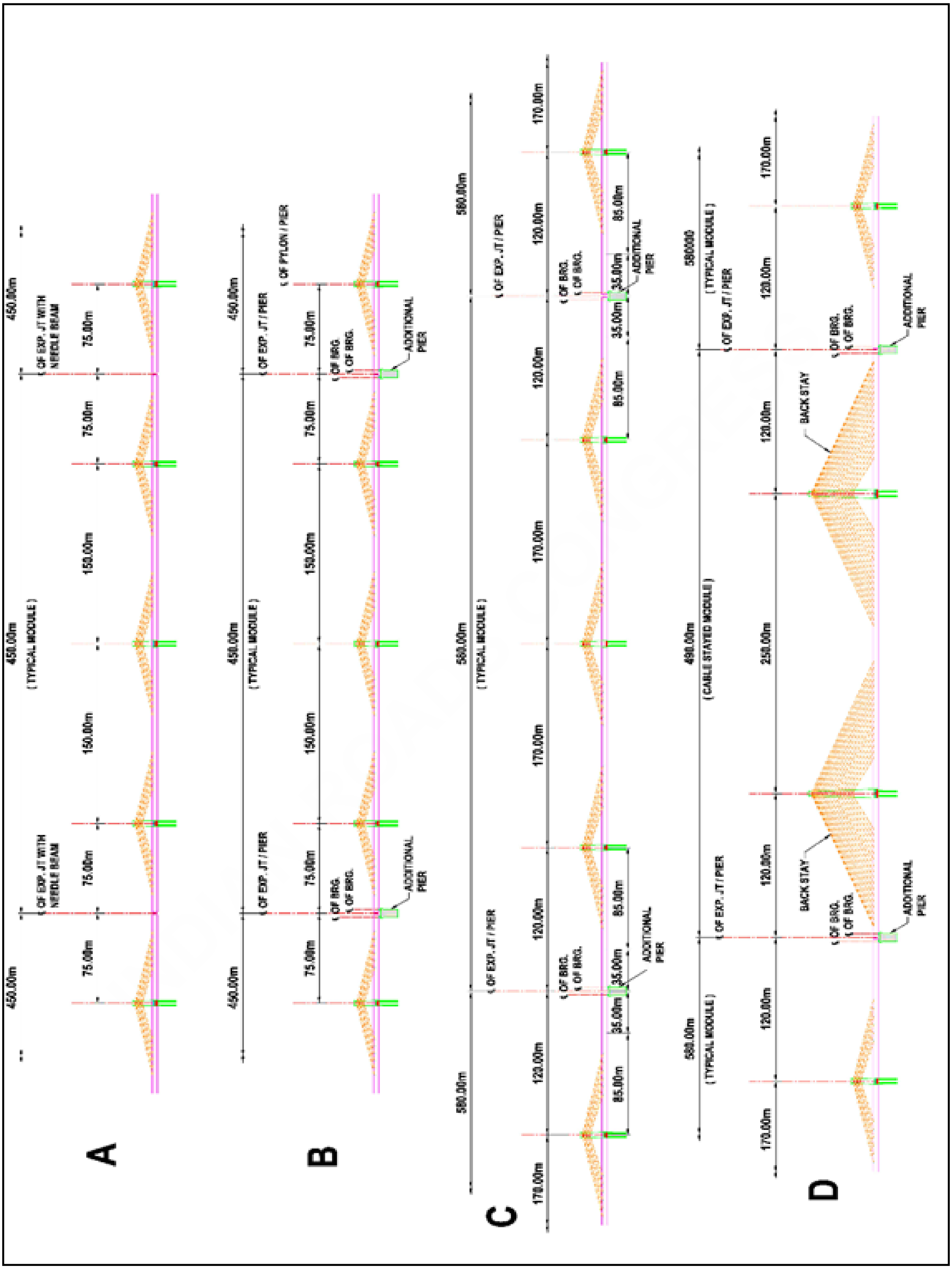


Fig 5.5 Structural options for Extradosed Bridges for Elimination of expansion joints at midspan

5.4.1 *Option A*

This is assumed as the basic commonly employed option (Fig 5) with the expansion joint at midspan concept for a typical project consisting of spans of 150 m and expansion joints located at 450 m c/c.

5.4.2 *Option B*

In the basic option discussed in 5.4.1, we could eliminate the expansion joints at midspans by introducing additional piers at every 450m. The resulting module would become 75+150+150+75=450m, which could be repeated for the full bridge length. The incremental cost, if any, of such an arrangement would be negligible. The number of bearings are reduced to 4 while the accessibility of the bearings and expansion joints, on the other hand, are much improved. This structural arrangement was first adopted for the Thane Creek Bridge, Mumbai (1995) taking account of the unsatisfactory performance of cantilever bridges with the old type of central hinge in the 70s and 80s.

5.4.3 *Option C*

A span arrangement of 120+170+170+120= 580m is an excellent option as it provides the traditional balance of keeping the end span as 0.7 times the main span. In case it is desired to alter (reduce) the spacing between the expansion joints, all span lengths can be reduced in the same proportion.

The unbalanced part of the cantilever, i.e., 120 -170/2=35m on both sides of the expansion joint (identified in Fig 5.5) can be constructed using a temporary formwork suspended in the end span as depicted in Fig 5.6. This arrangement was implemented more than 2 decades ago in the Jammu Udhampur Rail Link Bridges due to the enormous heights involved that made ground supported formwork impracticable.

5.4.4 *Option D*

The Inland Waterways Authority of India, (iwai.nic.in) is the national institution for the development and regulation of inland waterways for shipping and navigation in the country. They are the apex body for deciding the classification, minimum navigational clearances and characteristics of vessels plying in the concerned waterway. These norms are updated from time to time.

In a majority of cases the range of span lengths in which extradosed bridges are economical are also suited to the IWAI requirements. However, on occasion, larger clearances may have to be provided especially near maritime ports. In this case, a combination of Extradosed Bridge and Cable Stayed Bridge can be the most suitable option. In Fig 5.5 a cable stayed module of 120+250+120=490m has been interspersed in the extradosed bridge of Option C when a single 250m span meets the requirements. In some situations, more than a single main span becomes necessary when a multi-span cable stayed bridges can be provided, retaining the end spans as about half the main span to get an economical configuration. This arrangement was adopted for the Sultanganj Extradosed Bridge in Bihar.

Fig 5.6. Completing Unbalanced Portion of End Span of Free Cantilevering in Jammu Udhampur Rail Link with Span Arrangement of 70+102+70=242m. Height at Piers= 70M

5.5 Load bearing elements of extradosed bridges

5.5.1 *The concrete deck*

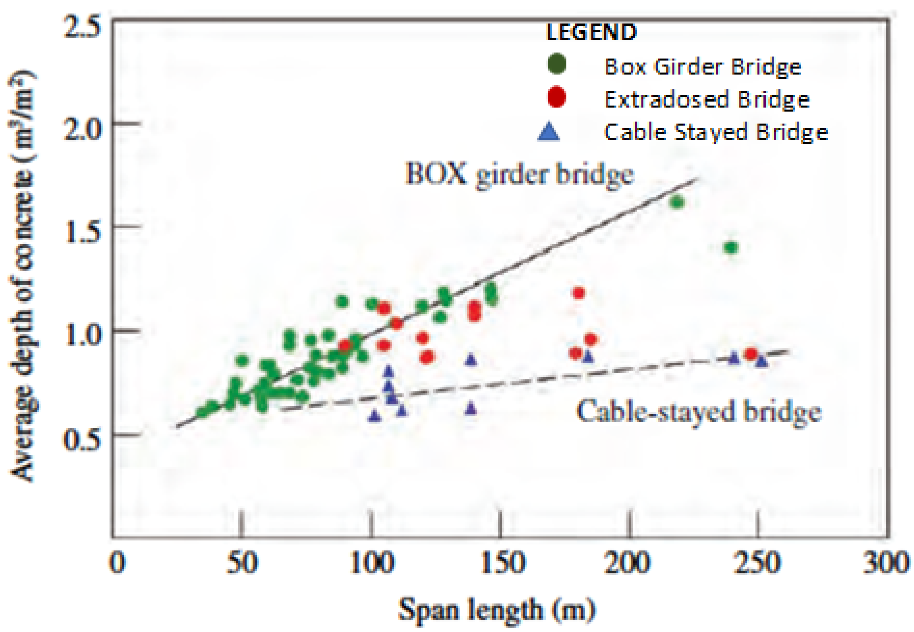


Fig 5.7 Estimate of Average Thickness of Deck Concrete [1]

Fig. 5.7 shows the concrete quantities for girder bridges, extradosed bridges and cable stayed bridges for various span lengths upto 250m. SED 17[1] gives the following approximate quantities per unit volume or area of concrete. Table 5.1 reproduces the SED 17 values, indicating quantities of Reinforcement and PT Tendons in Extradosed Bridges.

Table 5.1

Material	Mean	Coefficient of variation (%)
Reinforcement bars (kg/m³ in concrete deck)	168	22
Reinforcing bars (kg/m³ in pylons)	290	33
Post-tension steel (kg/m³ of deck area)	42	50

5.5.2 *Stay Cables*

Fig 5.8 shows stay cable quantities for single plane and two plane configuration of extradosed bridges. The wide difference of the two configurations are noteworthy. The single plane of stay cable configuration would of course put the onus of carrying the torsion due to eccentric loads on the carriageway entirely on the box girder section of the deck. SED 17 also includes quantities of single plane configuration for designs carried out by AASHTO code (Fig 5.9).

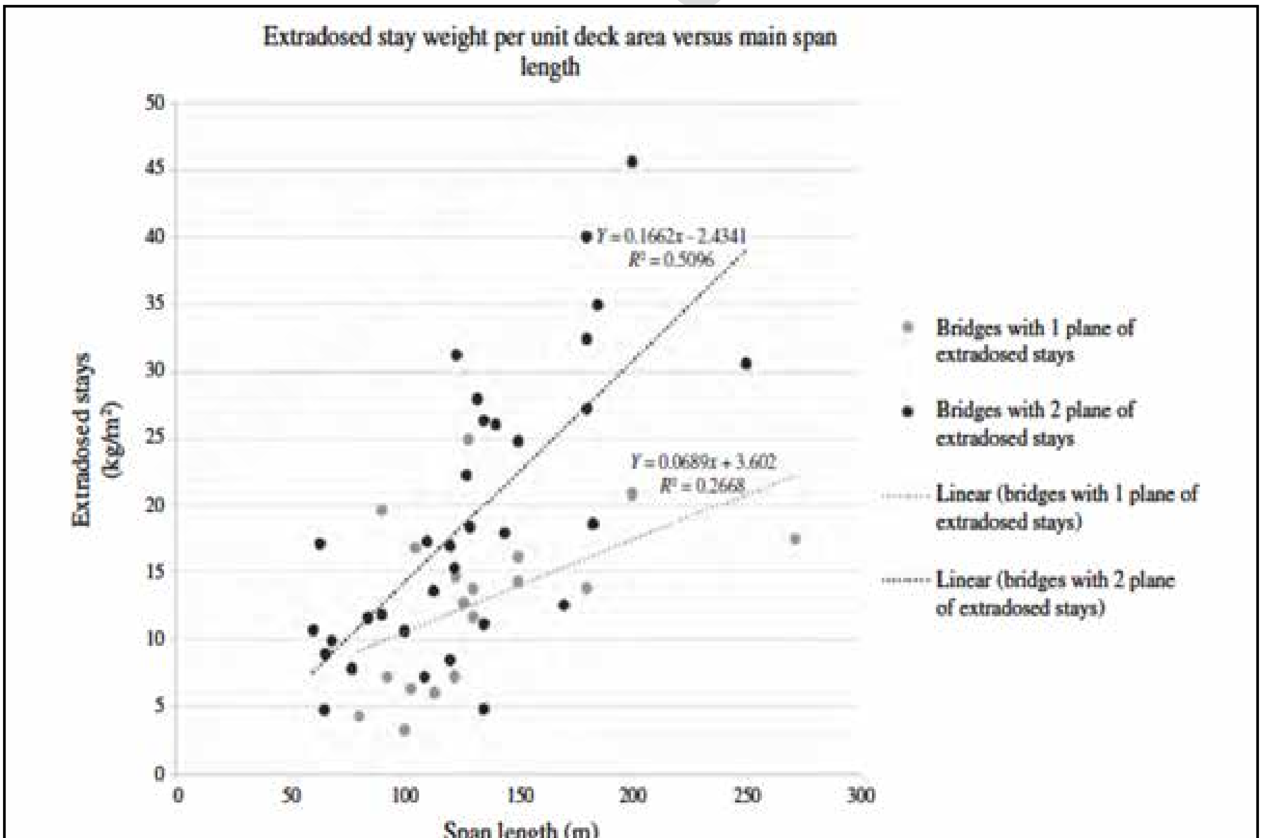


Fig 5. 8 Estimate of Stay Cables Based on Data Collected for 54 Bridges [1]

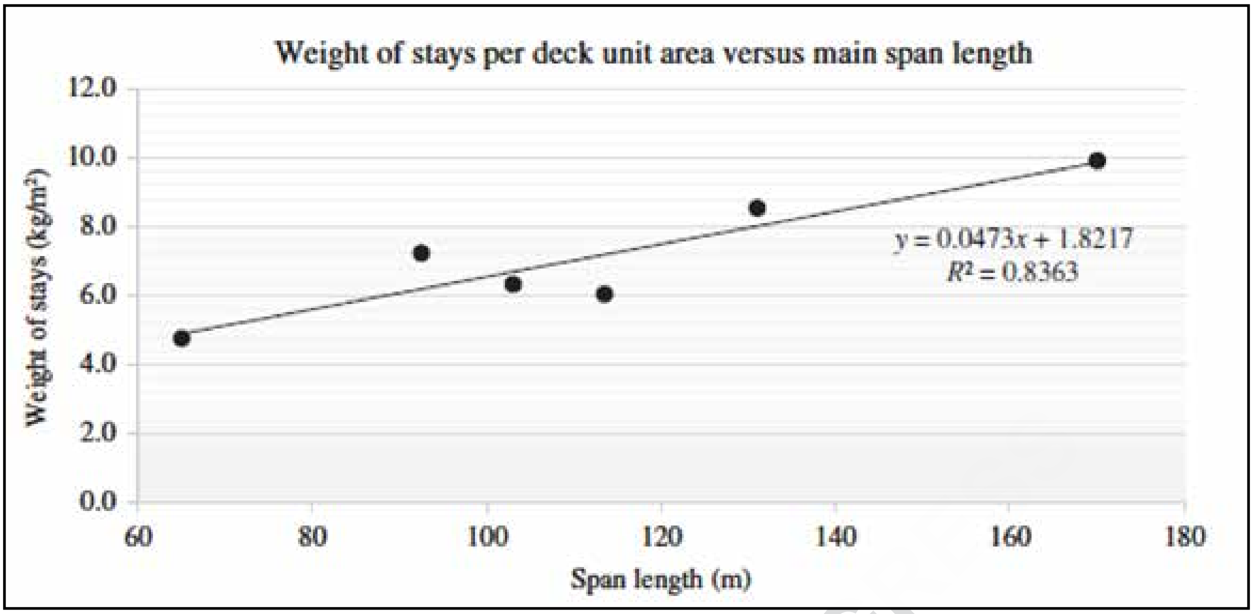


Fig 5. 9 Estimate of Stay Cables as per Aashto [1]

fib 89 [3], which is selected as the primary reference for these Guidelines makes a distinction between Cable Stayed Bridges and Extradosed Bridges for design and testing aspects. However, the document makes the distinction between the two types of bridges keeping in view their behavior only (refer Chapter 2) and does not propose any quantitative differentiation. The Extradosed bridge is characterized by a short tower height above the deck and a relatively stiff superstructure. On the other hand, the Cable Stayed Bridge is characterized by a tall tower and a flexible superstructure. This can also be seen in Fig 5.1.

The testing requirements for approval of the stay cable systems used in Extradosed applications are less stringent than those used in Cable Stayed Bridges and hence stay cable systems tested and approved for Extradosed Bridges are not intended for use in Cable Stayed Bridges. Chapter 6 of fib 89 gives detailed specifications of a variety of tests that are required for qualification of Stay Cable Systems for Cable Stay and Extradosed Bridge applications. More discussions on testing and on acceptance criteria are given in Chapter 8

Three Limit States are important from design considerations:

- SLS (Service Limit State)
- FLS (Fatigue Limit State)
- ULS (Ultimate Limit State)

fib 89 summarizes these in Clause3.2.1, Clause 3.2.2 and Clause 3.2.3 respectively. Chapter 7 of these Guidelines is devoted specifically to Loads and Load Combinations which relate to the above-mentioned Limit States.

International practice in France (Setra) and Japan are quite different with regard to design aspects. It is proposed to discuss these in the Guidelines for the purposes of information only.

The design of Stay cables is 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 this decides the permissible stress in the stay as a proportion of its ultimate load capacity. The permissible upper limit in the range of 40% to 45% of UTS is specified for Cable Stayed Bridges in the French [4] and Japanese codes [5]. In the case of a typical Extradosed Bridge the limit maybe as high as 60%. The stress changes due to live loads are low in Extradosed Bridges (as per [1]: 30 to 100 N/mm^2) as compared to cable-stayed bridges (up to 200 N/mm^2). The smaller stress changes in the cables due to live loads reduce the risk of fatigue and, therefore, the utilization of the cables can be higher than that applicable to cable stayed bridges. A value of 60% for Extradosed Bridge cables are specified in the Japanese Code, as shown in Fig 5.10

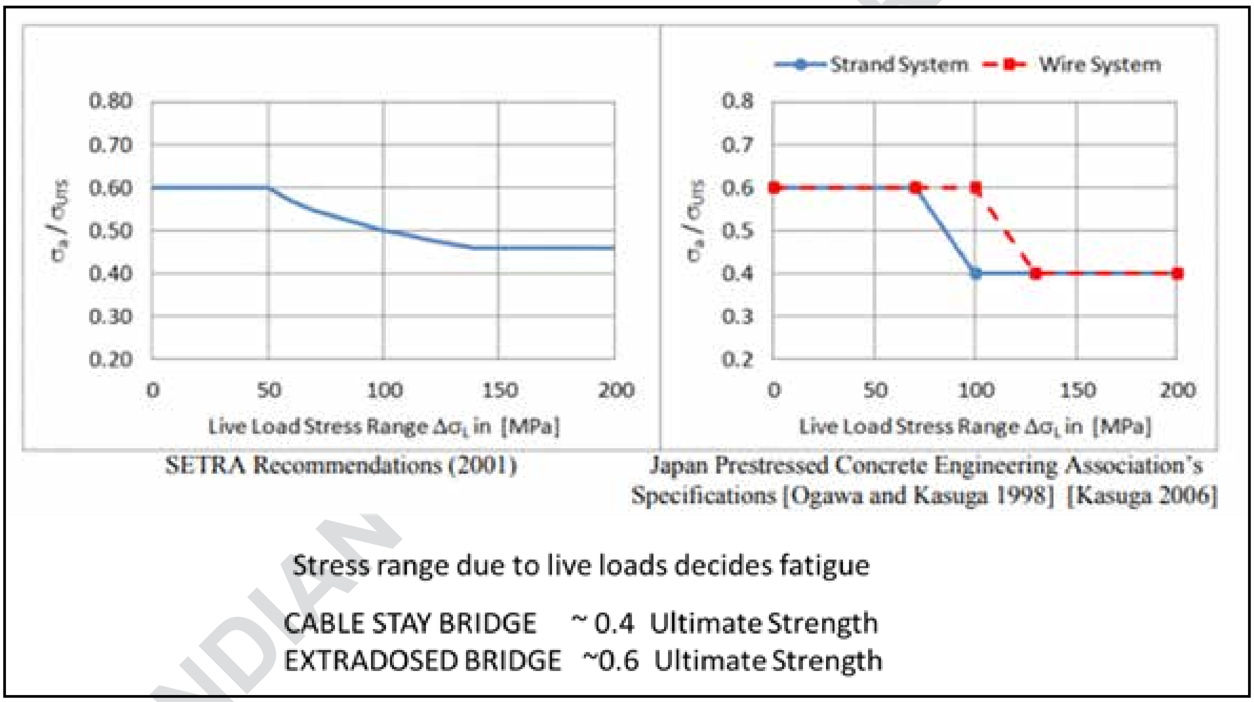


Fig 5. 10 Allowable Stress in Stay Cables in Setra and Japanese Codes Compared

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.

Ogawa and Kasuga in their paper "*Extrados Bridges in Japan*" (Ogawa, Kasuga, 1998 [6]) introduced the concept of the Beta factor which is defined as follows.

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

In a recent publication [7] Akio Kasuga has simplified the quantity estimation of stay cables for Extradosed Bridges as well as Cable Stayed bridges.

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%, Fig 5.11.

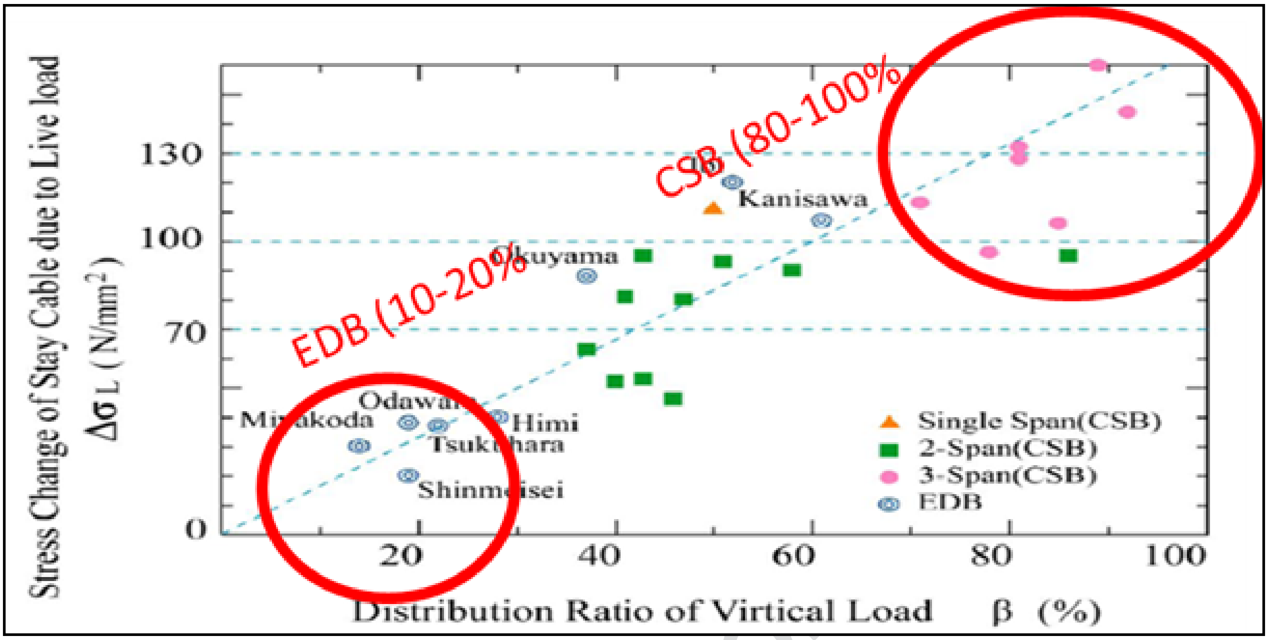


Fig 5.11. Beta Factor for Extradosed and Cable Stayed Bridges

In the recently published IABSE Bulletin SED 17 [1] the change of stress due to liveload and the corresponding max allowable stress is indicated in Table 5.2 for cable stays, extradosed cables and PT tendons. Whereas the max stress change is expected to happen in Cable stays almost negligible stress change occours in PT tendons. This should generally serve as a prelimnary check of FLS in the Cable Stayed and Extradosed Bridges.

TABLE 5.2

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

The corrosion protection requirements of stay cables are quite similar in Extradosed and Cable Stayed Bridges.

5.5.3 *Pylon*

Due to the short height of the pylon above the deck and large axial forces transmitted by the stay cables it is an aesthetical challenge to avoid making them look bulky and stocky.

In the case of a single plane of stays the pylon is merely an extension of the pier above the deck and its width in the transverse direction has to be made as small as possible so as not to occupy too much width of the deck. The upper part of the pylon is often flared out to accommodate the saddles or other arrangements of the stays which have a minimum radius of curvature from fatigue considerations. The shaping of the pylon as seen in the elevation of the bridge is sometimes the key to its aesthetic appearance .(Fig 5.12).

Fig 5.12 Extradosed Bridge at Moolchand, Delhi
Note shape of Piers and multi-coloured stays

In the case of two planes of stays the short height of the pylons precludes the possibility of joining them to form an "A" frame, which is possible and frequently seen in Cable Stayed Bridges.

5.6 Preliminary Design Calculations

5.6.1 *Step by Step Method for Preliminary Design*

Start with a good first assumed structural model so that only small refinements are required at predetermined points as the design proceeds and reaches its final shape. There are several publications based on past projects worldwide which can be used for the first model. These have been discussed in the foregoing already. This section describes the preliminary design of the superstructure subjected to vertical loads only using the computer equipped with general purpose or special software depending on the accuracy required and time available for the preliminary design. While wind is not usually important, earthquake effects must be considered and combined with other loads with appropriate load factors (see Chapter 7 for loads and load combinations).

Decide the broad parameters of the design by first defining the following:

a) Make a preliminary drawing of the bridge as seen in the elevation and section fulfilling the functional requirements of the project defined by the owner. *The elevation* should show the selected span arrangement, pylon height, as well as the location and type of bearings and expansion joints and configuration of stays, Fig 5.1 being a good starting point. *The section* should depict the complete deck width showing the carriageway as well as footpaths, median verge, cycle path, etc., as required. The depth can be initially taken as suggested in Fig 5.1. The single plane of stays will invariably require a box girder section of the deck as discussed earlier in the Chapter. The median verge is often a natural continuation of the provision in the approaches of the bridge. However, if the median verge is not available or not located at the centerline of the carriageway, two planes of stays become necessary. A worked example is given in the next section of this Guideline.

b) Define the Properties of Materials to be used in all the structural elements, including Deck, Stays and pylons. Estimate the weight of the deck (FIGS 5.9, 5.10) and calculate the SIDL

c) The approximate estimate of the quantity of Stay cables can be made on the basis of **Beta Factor** as explained earlier. A figure of 15-20% is a good estimation. Also, the change due to live load application should be checked, and keeping it within ~50 MPa as per SED-17, Table 5.2, is a good starting point. Another method could be to estimate the quantity of stays on the basis of the average inclination of stays as done in the Worked Example. This approximation can work well because the cables are concentrated in only 20% of the main span length as shown in Fig 5.1, thereby making the inclination of the cables within a short range.

d) Apply the SIDL which is generally mobilized after the free cantilevering of the deck has been completed. The wearing coat is invariably introduced at the final stage of construction. The stressing of stay cables is optimally done in two stages; the one during the construction of the deck while the dead load is being mobilised and the second after the SIDL has been provided. The second stage stressing should obviously be such that it caters also to the requirement for the live loads

Define Live Loading to be considered including Congestion Factor and SPV when applicable in accordance with IRC6 and carry out an initial run to find the enveloping effects of the same on all the structural components. This may be time consuming but it is an important step because live load effects are required in SLS and ULS checks as well as FLS checks for the stay cables for which fib 89 applies. They are also required for the design of the deck and pylon for which IRC 112 applies.

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CHAPTER - 6

CONSTRUCTION ISSUES

6.1 Introduction

For cost-effective design Extradosed Bridges are commonly built with main spans range of 100 to 250m and side spans of 50 to 150m. Precast or cast-in-place segmental construction is the most suitable construction method for these types of bridges which have been implemented in several projects. In general, the balanced cantilever method using segmental construction on both sides from the pylon is commonly adopted. However non-conventional erection schemes are also followed depending upon the site constraints. For example, the side span can be constructed on staging and the main span using the free cantilevering technique.

The method of erection is to be decided, considering the site constraints such as accessibility under the bridge, ease of transporting material, unbalanced force during erection, the viability of installing temporary supports, etc.

In 2007, Second Vivekananda Bridge, the multi-span and single plane cable supported Extradosed Bridge with six lanes of traffic was constructed using the conventional balanced cantilever erection method over river Hooghly connecting Howrah with Kolkata, in West Bengal, India (Refer figure 6.1). The bridge is provided with 7 spans of 110m and the deck width of 29m was constructed by using precast concrete segmental box balanced cantilever erection method. The precast segments were delivered by barges and lifted in place with a beam and winch system provided at the cantilever tip of erected segments.

Figure-6.1: Second Vivekananda Bridge public over Hooghly River [9]

Followed by the opening of the Second Vivekananda Bridge, many Extradosed bridges have been planned in India and they are under construction or successfully constructed with various ranges of spans, tower heights, and stay cable arrangements. The various construction techniques and methodologies for each component of these bridges are described in this chapter.

6.2 PYLON

6.2.1 *General*

In general, the pylons are constructed prior to the construction of the superstructure regardless of the erection scheme of decks such as full staging or balanced cantilever segmental construction. After the construction of the pylon, the girder will be constructed supported by the lower pylon and inclined cables. It is evident that the final construction stage forces of Extradosed bridge elements such as pylon, girder and cables are related to the erection method and sequence. Therefore, it is important that the construction methodology and erection sequence should be considered in the design of the Extradosed Bridge. It is also very important for the designer/contractor to understand the structural behavior of each element giving rise to forces and deformation at each construction stage and ensuring the design adequacy and structural stability during the construction stage.

6.2.2 *Lower Pylon Construction*

In principle, the construction of a lower pylon is similar to the construction of building cores and chimneys. In general, the lower pylon is built cast-in-place as a reinforced concrete structure. Pour height, i.e. lift height, and the number of pours should be decided depending on the construction planning and formwork design.

Use of a self-climbing formwork system such as jump-form or slip-form is the most suitable methodology for the construction of the tall and vertical structure.

Jump-form is progressing to the next pour lift after the concrete in the previous lift has achieved the strength required to retain the form. This can be very productive and fast minimizing the labour work at the site. This formwork can be lifted off and reattached with various methodologies such as by using the crane, rail, or hydraulic system.

Slip-form is a continuous pour system involving a self-climbing formwork that supports itself on the concrete poured previously and achieved strength, moving slowly over the concrete as it is cast in a continuous, monolithic pour.

The use of Jump-form is commonly preferred in order to ensure any built-in items are in the right position.

(i) Rebar assembly (ii) Installation of Form work (iii) Finished Concrete

Figure-6.2: Lower Pylon Construction [9]

6.2.3 *Casting of Pier Table*

Another important activity in bridge construction especially which plays a key role when the balanced cantilever construction method is adopted is the casting of pier table. Due to significant self-weight full staging, rebar arrangement and concreting is the traditional way of constructing the pier table. Pier table construction starts after the completion of the lower pylon.

Alternatively, the pier table can be constructed partly with precast-like shell with a supporting steel frame and partly with cast-in-place concreting after erecting the precast pier table shell. A temporary bracket support is erected on the top of the lower pylon. This acts as a support for the erection of the pier table with a partial precast shell. After the erection of the pier table, further construction activities such as assembling of shutters, tying of reinforcement and concreting commences. This concludes the casting of the pier table operation with lesser time duration than the conventional cast-in-place concreting.

Figure 6.3.(a), shows the construction activity of full-stage cast-in-place concreting of the pier table and Figure 6.3.(b), shows Partial precast pier table shell with supporting steel frame and cast-in-place concrete from different bridge projects.

(i) Bracket Erection (ii) Installation of Form work (iii) Finished Pier Table

Figure-6.3.(a): Full Stage Cast-in-place Pier Table [9]

(i) Shifting Pier Table Segment (ii) Erected Pier table segments (iii) Finished Pier Table

Figure-6.3.(b): Precast Pier Table [9]

6.2.4 *Upper Pylon Construction*

After completing pier table construction, making of upper pylon position should be carried out by survey from permanent reference benchmarks. After fixing survey alignment for pylon, reinforcement should be assembled and stay cable anchorages should be embedded for every lift of pylon construction. The formwork should be started once reinforcement tying and aligning is completed. The shutter is either of structural steel fabricated to suit the pylon profile (or) Ply formwork. Pylon is cast in stages using the formwork. Formwork is prepared separately for pylon up to cable locations and in cable locations. In general, the formwork is shifted between lifts by using Cranes, Tower cranes or a hydraulic system. The shuttering is supported on cantilever brackets with a working platform for placing the concrete. Concreting can be done with concrete buckets held using the tower crane or using the boom placer based on the maximum height of the upper pylon. The number of pours should be decided based on the upper pylon height and the design of the formwork. Installation of Prefabricated saddles for stay cables using sacrificial support is preferred as it is easy to ensure the saddle system is aligned as per the required geometry.

(i) Full Trestle Support (ii) Jump Formwork

Figure-6.4: Upper Pylon Construction [9]

Figure-6.5: Tower Crane for Pylon Construction [9]

6.3 SUPERSTRUCTURE

6.3.1 General

The following is a summary of the construction methodologies for concrete segmental bridge structures being widely used in the industry today. Each methodology has unique features and unique advantages that contribute to its most appropriate use. The selection of which type to be used for a particular project depends on the location of the structure, access to the site, environmental considerations, desired span lengths, availability of precast yard location, and ability to deliver materials to the site.

The various methods of deck construction are:

a. Construction of Cast-In-Place on Full Staging

b. Construction of Precast on Full Staging

c. Construction of Cast-In-Place Balanced cantilever erection

d. Construction of Precast segmental Balanced cantilever erection

In designing an Extradosed bridge as a segmental bridge, it is important to pay attention to the details listed below, which may facilitate the project to be more cost-effective, easier to construct and have better quality control. The following guidelines apply to both cast-in-place and precast construction

a. Keep the length of the segments equal, and keep the segments straight even for curved structures (chord elements).

b. Maintain constant cross-sectional dimensions as much as possible. Variations of cross-sectional dimensions increase the cost and cycle time of segment production. Hence it is advised to minimize the variable dimensions unless not avoidable such as a change in the depth of webs and thickness of the bottom slab.

c. Segment proportions (e.g., shear keys) should be such as to allow easy form stripping.

d. Avoid as much as possible surface discontinuities on webs and flanges caused by anchor blocks, inserts, etc.

e. Use a repetitive layout for tendons and anchors, if possible.

f. Avoid dowels passing through formwork, if possible.

The various construction methodologies adopted for erection are illustrated below:

6.3.2 *Construction of Cast-In-Place Full staging*

This method is considered when there is a variation in width of segments and precast is not feasible, (or) when there are constraints for precast segment transportation and feeding.

Formwork for cast-in-place decks should be supported by a temporary staging arrangement. Formwork along with its temporary supports and foundations must be designed to safely support all loads that might be applied without undesired deformations or settlements. Soil stabilization of the foundation may be required depending on the soil strata.

Formwork for structures of variable geometry should need to be relatively flexible in order to allow adoption at the various joints. Both external and internal forms are usually retractable in order to provide a free working space for placing reinforcing steel and prestressing ducts.

Special consideration must be given to those parts of the forms that have variable dimensions.

To facilitate alignment or adjustment, special equipment such as turnbuckles, prefitted wedges, screws, or hydraulic jacks should be provided.

Figure-6.6: Concrete deck on temporary continuous or discrete propping [9]

6.3.3 *Construction of Precast full staging*

Erection of the precast segments on scaffolding may be considered in case the bridge deck is constructed on the land and the level of the deck is not too high so that segments can be lifted and positioned by using a crane. Each segment is placed usually on a steel trolley as that shown in Figure 6.7. The scaffolding should be provided with hydraulic and mechanical jack systems to allow the trolley to be moved along the precast segment in the longitudinal and transverse directions to reach the required alignment in the plan. The trolley itself should be provided with hydraulic and mechanical jacks to allow the segment to move upwards or downwards, relative to the trolley, to occupy the required alignment vertically.

Figure-6.7: Erection of precast segments on scaffolding [5]

Prefabrication of segments normally takes place off-site, and units are erected in lengths of 3 to 4m, based on the transportation feasibility and erection capacity of cranes. The length of free cantilever possible during the construction phase depends on the deck characteristics and must be carefully determined for the temporary conditions though un-propped lengths of over 15m have been successfully achieved. On completion of the deck, all the stays are connected, and tensioned and the temporary supporting towers are then dismantled. However, some extension of the cable is unavoidable due to the self-weight of the deck. The temporary propping should therefore be erected at a height calculated after allowing for this movement.

6.3.4 *Construction of Cast-In-Place Balanced cantilever erection*

This method of bridge construction has been divided into two parts including cast-in-place post-tensioned and free cantilever erection. Post-tensioning is a technique in which it pre-loads the concrete and restrains the tensile stresses induced in the girder due to dead, live loads and thermal effect, etc. In this method, high-strength concrete as well as high-strength post-tensioning strands are used. After concrete gains strength, the strands are pulled by a special jack to a pre-determined force and locked.

This method has several advantages namely reducing direct cost, reducing the required material, rebar, transportation, and casting yard costs. And further, this method increases the construction efficiency and better structural performance.

The balanced cantilever method is the second part of the cast-in-place classification which has been considered the most effective method in bridge construction. The most advantage of using this method, especially in urban areas refers to its characteristic that it does not need any temporary shoring and it allows traffic flow below the decks or avoids distrupting water channels under the bridge. Normally the length of each segment in this method should between 3-6 meters. This method usually starts once pier construction is completed and is followed by erecting segments on both sides of the pier simultaneously and each segment would be tied to the previous segment by post-tensioning tendons.

Figure-6.8: Erection of cast-in-place segments on scaffolding (Ah-am Bridge in S. Korea) [6]

6.3.5 *Construction of Precast segmental balanced cantilever bridges*

The occasional need to have clear uninterrupted space below the bridge, for example railway sidings, private property, etc., has forced to develop the balanced cantilevering technique, where no props are required to be installed.

Figure-6.9: Balance Cantilever Erection of Precast Segment [9]

In general, erection proceeds simultaneously on each side of the tower minimizing the unbalanced force at the lower pylon as shown in figure 6.9 and 6.10. It would be the most ideal and economical solution as the lower pylon would not experience significant unbalanced force during the construction even after completion of the bridge due to self-weight of deck.

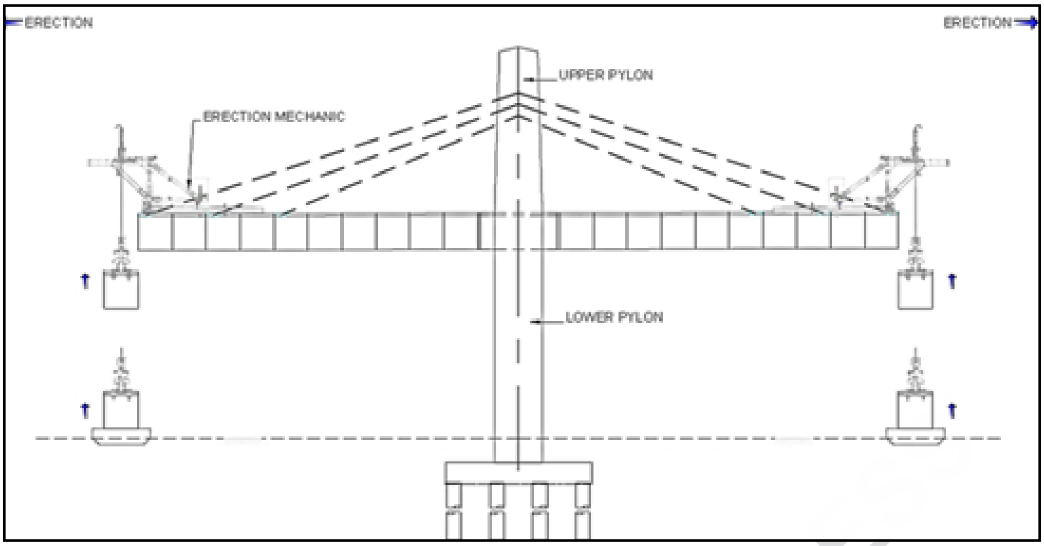


Figure-6.10: Balance Cantilever Technique [9]

Typical construction scheme using precast segmental balanced cantilever erection is shown in the figure 6.11.

<u>Construction Methodology with proposed for New Ganga Bridge Project, Patna i.e. Six lane extradosed bridge over River Ganga</u>

Precast segmental balanced cantilever erection method using derrick crane is adopted for construction of typical block of main bridge for New Ganga Bridge Project over River Ganga. The pier table is constructed using cast-in-place method over temporary support system on lower pylon. The precast segments of the deck are fabricated at facility yard first, then transported to the site and erected by the derrick crane. After lifting and aligning the precast segment precast segment is glued to the previous segment by using the epoxy and temporary post-tensioing bars. Permannent post-tensioning and extradosed staycable is installed in sequence. Key segment is installed in the intermidante span and need beam is installed in the expansion joint span. The construction scheme of the main bridge is illustrated in the sketches below.

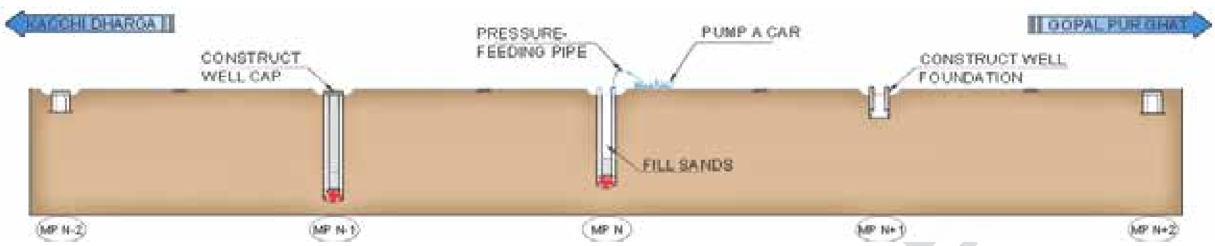
[Phase 1 : Foundation Construction Stage]

1. Construction of well foundation (Sinking and Concreting for steining is staggered process)

2. Closure with Bottom Plug,

3. Filling material inside,

4. Well Cap concreting.

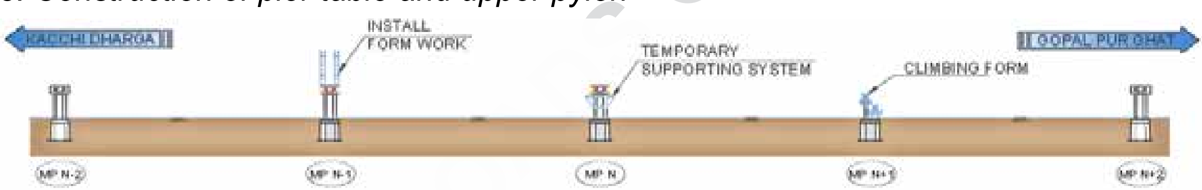


[Phase 2 : Pylon Construction Stage]

1. Fabrication of reinforcing bar and casting concrete for lower pylon.

2. Install the temporary support system for pier table.

3. Construction of pier table and upper pylon



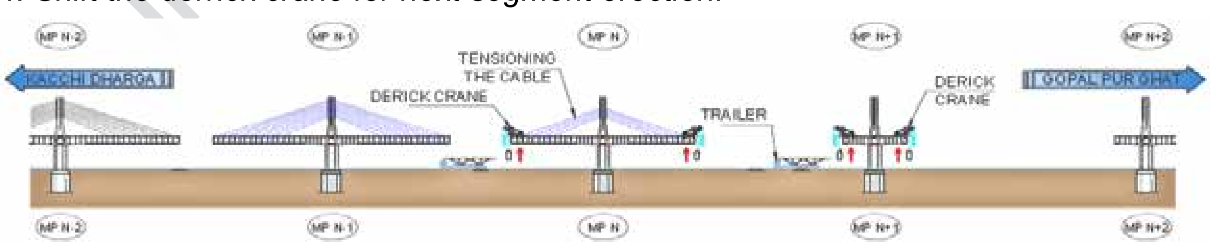
[Phase 3 : Segment Erection Stage]

1. Transport segments by trailer from casting yard to erection location.

2. Lift the segment by derrick crane and connect to previous segment

3. Apply prestressing of tendon and extradosed cables in due sequence.

4. Shift the derrick crane for next segment erection.



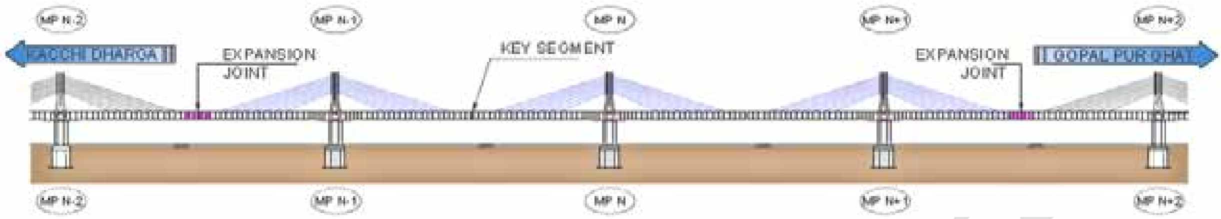
[Phase 4 : Key Segment Closure Stage]

1. Lift key segment formwork by derrick crane.

2. Lift internal movement joint.

3. Construct key segment.

4. Install internal movement joint (Needle Beam)



[Phase 5 : Finishing work] *1. Install the expansion joint. 2. Install wearing surface and road facilities*

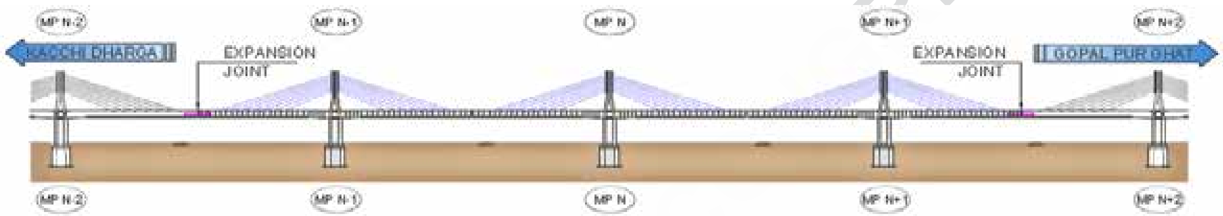


Figure-6.11: Typical Construction Scheme of Precast Segment Balanced Cantilever Erection [9]

Although symmetric balanced cantilever erection is most ideal and economical way of construction for precast segmental Extradosed Bridge, construction of asymmetric span bridge erection is also technically feasible and constructed successfully by stiffening the back span girder and balancing the back span weight as shown in figure 6.12.

Figure-6.12: Construction of balanced cantilever construction for

asymmetric main span [9]

Stability during the erection against unbalanced self-weight, construction load, equipment load as well as fluctuation of wind load should be ensured in design and specified in the drawings. Details required to ensure the stability should be implemented during the construction as specified in the design. Monolithic connection between pylon and girder can be easily detailed in such a way for achieving the required strength against the unbalanced force anticipated during the construction. However in case the lower pylon is not designed sufficient to resist the construction stage unbalance loads, stability should be ensured by providing additional stability towers etc.

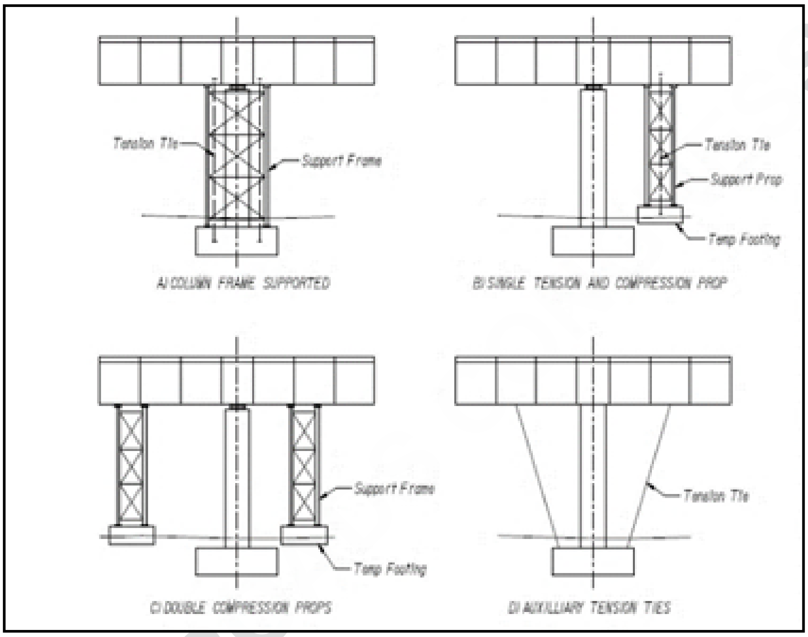


Figure-6.13: Methods of additional stability [1]

In case of bearing connection between superstructure and substructure adopted, stability of the structure should be ensured during erection of segments, either by providing temporary stability towers (bents) or connecting the pier table with pier cap.

6.3.6 *Production methods of precast segments*

Precast segments are usually cast using Long line or Short line method of casting. The method of casting should be chosen based on the segment complexity and space availability in the casting yard and the overall cycle time for the segment production.

6.3.6.1 *Long-Line Method*

In this method, Mould is arranged such that complete bottom form (soffit), which is fixed and has the same total length as the entire span. The soffit is made in the profile of the structure with correction for short and long term erection deflection. The external and inner forms having lengths of a single segment are also moved along the run. In this method most of the geometry control is done during the preassembly of the soffit, reducing the need for geometry control during each segment production. Segments remain in their relative positions, which makes geometry control simpler. Ground support should be stiff enough so that there is a minimal settlement for any segment and should not affect the achieved geometry. Figure 6.14 shows the typical production line for long line method with casting and removal sequence. Figure 6.15 and 6.16 shows the casting yard operation for a project with long line method.

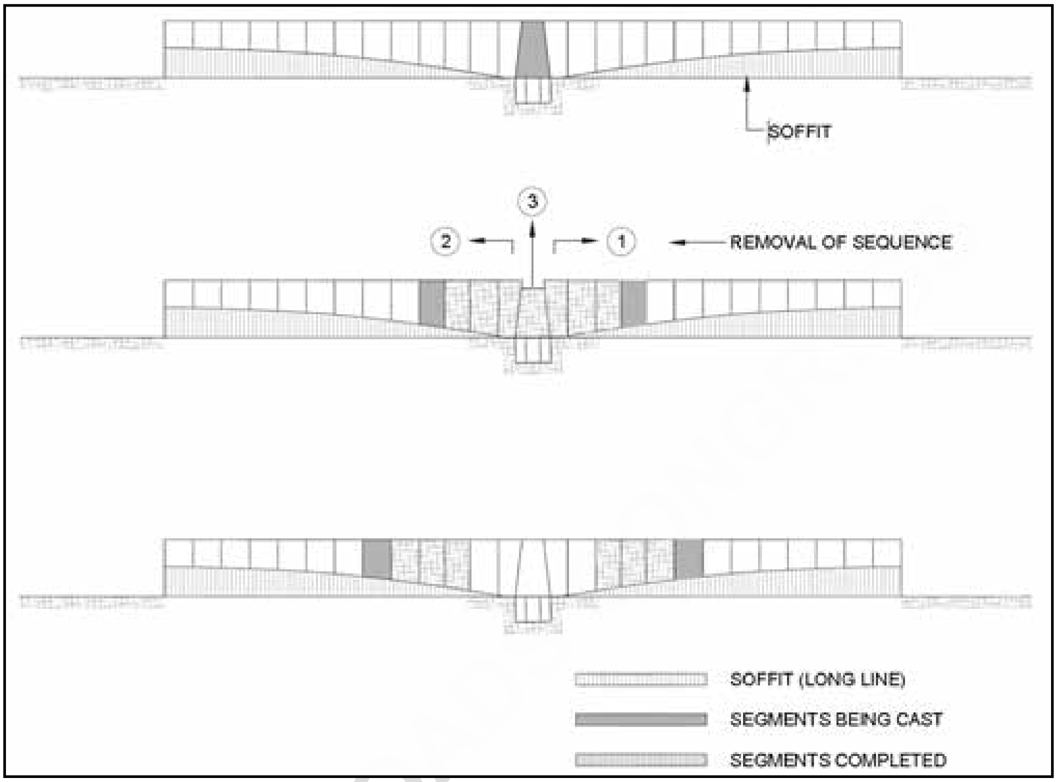


Figure-6.14: Typical Long-Line Production [1]

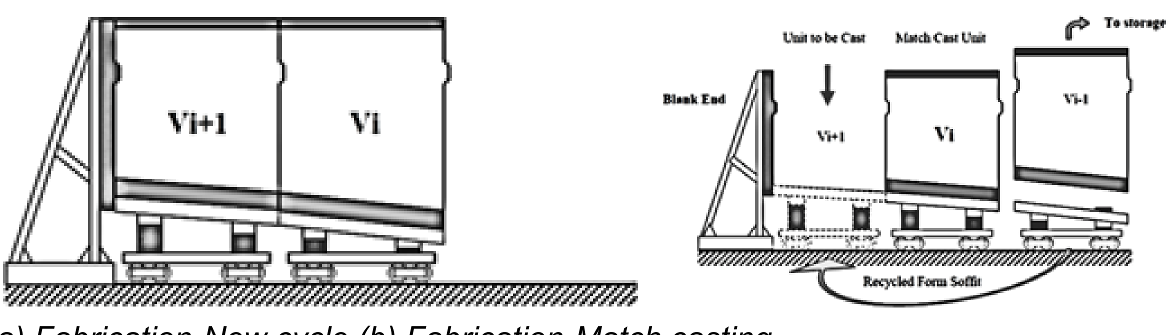
Figure-6.15: Long-Line method [9]

Figure-6.16: Long-Line method [9]

6.3.6.2 *Short-Line Method*

In this method, Mould for one segment casting is provided. The casting is done in sequence from one side of the span to other end of the span i.e. Mould is stationary and segments are moved from wet cast position to match casting position, and then to the stacking area. By using this method space requirements can be reduced as entire casting process is centralized. This method is extremely adaptable to geometric variations such as horizontal and vertical curvature and superelevation transitions, which are obtained without increase in cost compared to the Long-Line method.

For vertical curve, match cast segment is first translated from its original position and then a small rotation to given in verical plane with respect to horizontal axis and for horizontal curve, segment is first moved to its position by pure translation and then segment is rotated in plan with respect to the vertical axis as shown in Figure 6.18



(a) Fabrication-New cycle (b) Fabrication-Match casting

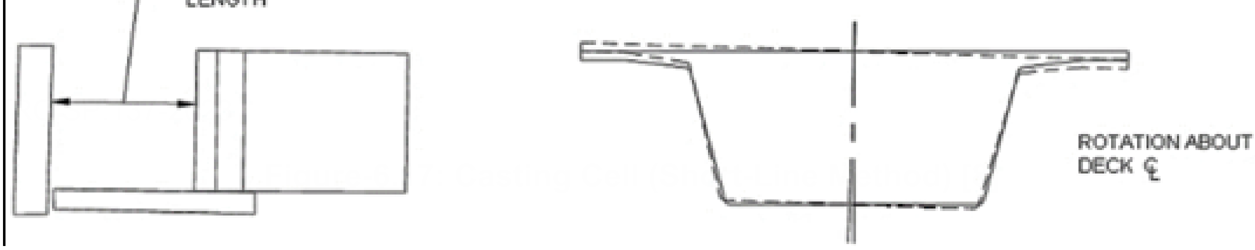


Figure-6.18: Form Adjustment in Casting Cell (Short-Line Method) [1]

Figure-6.19: Short-Line Form [9]

6.3.7 *Cable Stay System*

6.3.7.1 *Installation and Stressing of Cable Stays*

6.3.7.1.1 *Stay Pipe Erection*

The stay pipe is lifted with the first strand to be installed which is connected at both ends to anchorages and stressed. The stay pipe is then raised for erection using a tower crane and secured to the pylon.

6.3.7.1.2 *Strand Installation – Initial Stressing of Stay*

One or two strands that are connected to anchorages are pulled through the stay pipe and then stressed individually, using an automatic, computer-controlled system that ensures that the strands are parallel.

6.3.7.1.3 *Strand Installation – Final Stressing*

Strand installation methods are engineered to avoid any kind of cable de-tensioning. All the stressing operations, including fine-tuning, are carried out using an automatic stressing mono strand jack. A compact multi strand jack is used only when final stressing/ de-tensioning is unavoidable.

Figure-6.20: Cable Stay System [10]

The strands are often tensioned one by one thereby eliminating the need for heavy pre-stress equipment. Though the method has several variants depending on the patent used by each company, it basically involves pre-stressing starting with a single strand up to the defined stress level. Once the strand is anchored, the rest of the strands are successively pre-stressed one by one until their stresses match the stress of the first one which could, however, lose its tensile stress when the other strands are pre-stressed, due to the elastic shortening. This aspect must be considered in the design of tensioning process along with the fact that often the first strand is carrying the weight of the cable duct on its own. (Figure 6.21).

Right through the process of structural analysis of the construction process, temperature is usually assumed to be constant and equal for all structural elements. However, at every stage, it is possible to find differing temperatures in various parts of the structure on the site depending on their different thermal properties and colors, because pre-stressing operations are performed throughout the day. Although this phenomenon is often ignored by the contractor, there is no doubt that taking cognizance of it will reduce the differences between the designed forces and actual ones at the end of the pre-stressing process.

Anchorages usually allow for a global re-tensioning of the full stay. However, heavy jacks and oil pumps are required and, moreover, tolerances taken must be considered, especially for short stays. A site engineer therefore must carefully design the initial stress of the first strand to avoid re-tensioning operations.

Figure-6.21: Stay duct weight born by the first strand in the strand by strand tensioning technique [10]

6.3.7.1.4 *Cable Erection*

These days, the majority of the extra-dosed bridges are designed with parallel strands types. The simplest erecting procedure is to unreel the strand along the deck and hoist or lift it up to the top of the pylon. Unfortunately, the natural sag tends to be quite large, and therefore considerable take-up must be provided in the tensioning jack. A more satisfactory procedure is to install a guide rope and pull the cable up with a hauling rope. Intermediate supports to reduce sag must be provided through intermittently spaced sliding hangers. Tensioning is initially carried out at the deck connection end to take up the slack and final tensioning is to relieve the bending moment in the deck and transfer the dead load onto the cable after all work on the newly erected section is complete (i.e. post-tensioning of concrete segments, etc.).

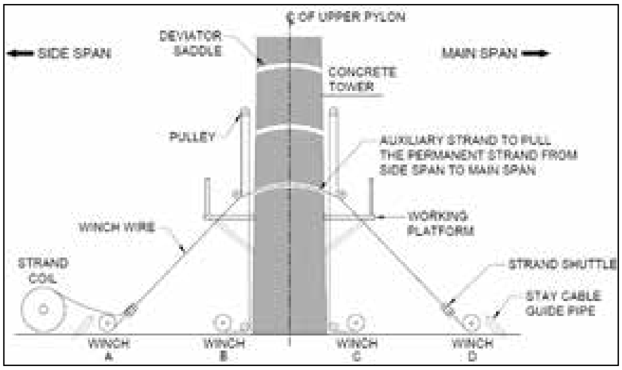


Figure-6.22(a): Threading the strands from the Anchorage location on the side span side to the Anchorage location on the main span side through The deviator saddle (schematic) [5],

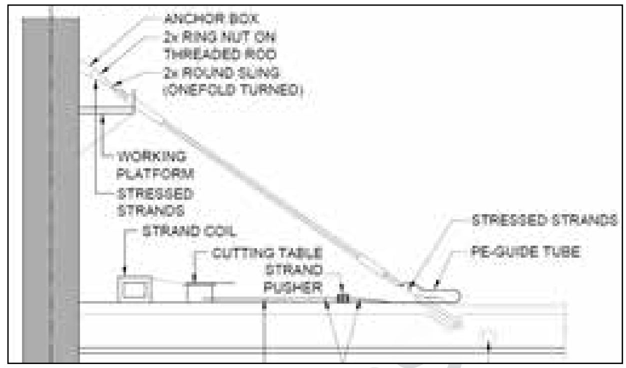


Figure – 6.22 (b): Threading the strands for anchor box type system

6.4 Geometry Control

Geometry control is an essential part during construction to achieve the required profile and alignment. It is important to control camber (geometry control) to enhance maintenance and driving comfort during service life. It is a process flow and engineering assessment in order to match the final geometry of the structure and it is carried out by performing structural analysis with a predefined construction scheme considering actual construction conditions i.e. effect of temperature, working load, actual equipment load, and time effect due to creep and shrinkage etc.

Geometry control of Extradosed Bridge erected by balanced cantilever construction method provides displacement of the girder and stay cable forces at each construction stage. Geometry control is performed in such a way that stay cable stresses, displacement, and girder stress do not exceed the target tolerance during the entire construction and the intended limit in the final constructed bridge.

Sensitivity analysis is recommended to be carried out by Geometry Control Engineer separately to understand the impact on the structure due to various parameters as indicated below for better assessment of the geometry control during construction

a. Seasonal variation of temperature (Temperature uniform) and Daily variation of temperature (Temperature gradient)

b. Elastic modulus of concrete

c. Time delay between two cantilevers (closure connection)

d. Actual segment weight and derrick crane/Form traveler weight as per site condition

e. Density of concrete

f. Unit load assessment on the stay cable and cantilever tip

g. Foundation Settlement

6.4.1 *Geometry Control for Cast-in-Place (CIP) Construction*

Cast-in-Place balanced cantilever construction refers to a construction process where segments are progressively cast on-site at their final position in the structure with use of Form Traveler (F/T). The bridge segments are cast-in-place, in progressive increments and one segment at a time. Formwork of the segment of the previously cast remains in place until the new segment has achieved sufficient strength to be post-tensioned and permanently held in place by previous cast segments behind it.

Geometry control adjustment procedure for cast-in-place construction differs slightly from precast construction. In cast-in-place construction, local adjustments are made with the traveling forms instead of by shimming the joints which is in the case in precast match cast. Geometry discrepancies due to additional segment weight or inaccurate stay cable jacking forces still require re-stressing of the stay cables.

Tolerances in cast-in-place segmental bridges should be adhered to with great care. A form which produces heavier segments on one of the cantilever arms will have a large effect on the unbalanced moment. The geometry control for cast-in-place segmental bridges is less demanding since it is almost "self-correcting". Curing on the other hand is more critical since the progress of the work depends on the concrete achieving early strength. Post-tensioning generally occurs long before the concrete achieves design strength and anchor sizes may have to be adapted for lower strength of concrete. The sequence of segment concreting has to be adapted to suit the flexibility of the form carrier.

Figure-6.23: Form traveler erection arrangement [6]

Factors on influencing the camber

a. Loads applied to the structure: Self-weight of segment, weight of bridge builder / derrick crane, pre-stressing, temperature, erection load, construction live load and stay cable forces

b. Material property and characteristics: elastic modulus of concrete mix, concrete creep and shrinkage, elastic modulus of stay cables including non-linear effects due to sag, elastic modulus of tendons and relaxation.

c. Deflection of Form Traveler (F/T)

d. Construction schedule and sequence, including the erection sequence

In practice, the most critical problem of cast-in-place construction is deflection control, particularly for long-span structures. There are five categories of deflections (or geometrical movements in space of the structure) during construction and after completion:

a. Deflection of the travelers under the weight of the concrete segment. This value is given by the manufacturer or may be computed and checked at the site during the first operation.

b. Deflection of the concrete cantilever arms during construction. For each casting of a pair of segments, the weight of the concrete segments and the corresponding cantilever prestressing forces impose on the cantilever give raise to a new deflection curve.

c. Deflections of the various cantilever arms after construction and after removal of the travelers before continuity is achieved with the other parts of the deck.

d. Short-term and long-term deflections of the continuous structure, including the effect of superimposed dead loads (curbs, railings, pavements, utilities, and so on) and instantaneous deflection due to live loads

e. Short-term and long-term pier shortenings and foundation settlements.

Using the data available on concrete properties and foundation conditions, the designer should compute the various deflections mentioned under items c, d, and e of above, assuming that the bridge super structure will be loaded due to foundation settlements also and the long-term and short-term concrete deflections will occur including the foundation settlement. The sum of the various deflection values obtained in the successive erection of the deck allows the construction of a camber diagram, which should be added to the theoretical longitudinal profile of the bridge to determine the casting curve for each cantilever arm. This casting curve is the goal, towards which construction proceeds during cantilever casting. Short-term and long-term deflections of concrete (pier and foundation) can be adjusted during the construction of pier by altering the pier height. Settlement of foundation should be calculated based on the theoretical properties of soil and should be closely monitored during construction. And if any settlement happene during construction, Geometry Control Engineer should suggest the remedial measure to adjust the settlement during erection.

6.4.2 *Geometry Control for Precast Segmental Construction*

In balanced cantilever precast segmental construction, erection progresses outwards from central pier until adjacent cantilevers meet at mid-span with closure connection. Segment erection generally progresses in symmetric pairs, with one segment of each pair secured to opposite ends of the previously completed structure with temporary post-tensioning bars. Each pair is stressed against the structure with cantilever tendon that are located in the top flanges of the segments and run from one side of cantilever tip to the other side of the cantilever tip.

Geometry discrepancies due to casting errors or inaccurate placement of first cantilever segment can be controlled by adjustment procedure with application by shimming the joints. Geometry discrepancies due to additional segment weight or inaccurate stay cable jacking forces still require re-stressing of the stay cables depends upon discrepancy magnitude

Factors influencing the camber

a. Loads applied to the structure: Self-weight of segment, weight of bridge builder / derrick crane, pre-stressing, temperature, erection load, construction live load and stay cable forces

b. Material property and characteristics: elastic modulus of concrete mix, concrete creep and shrinkage, elastic modulus of stay cables including non-linear effects due to sag, elastic modulus of tendons and relaxation.

c. Geometric deviations incurred during casting

d. Construction schedule and sequence, including the erection sequence

e. Temporary Pre-stressing application, deviations incurred during epoxy matting of segments

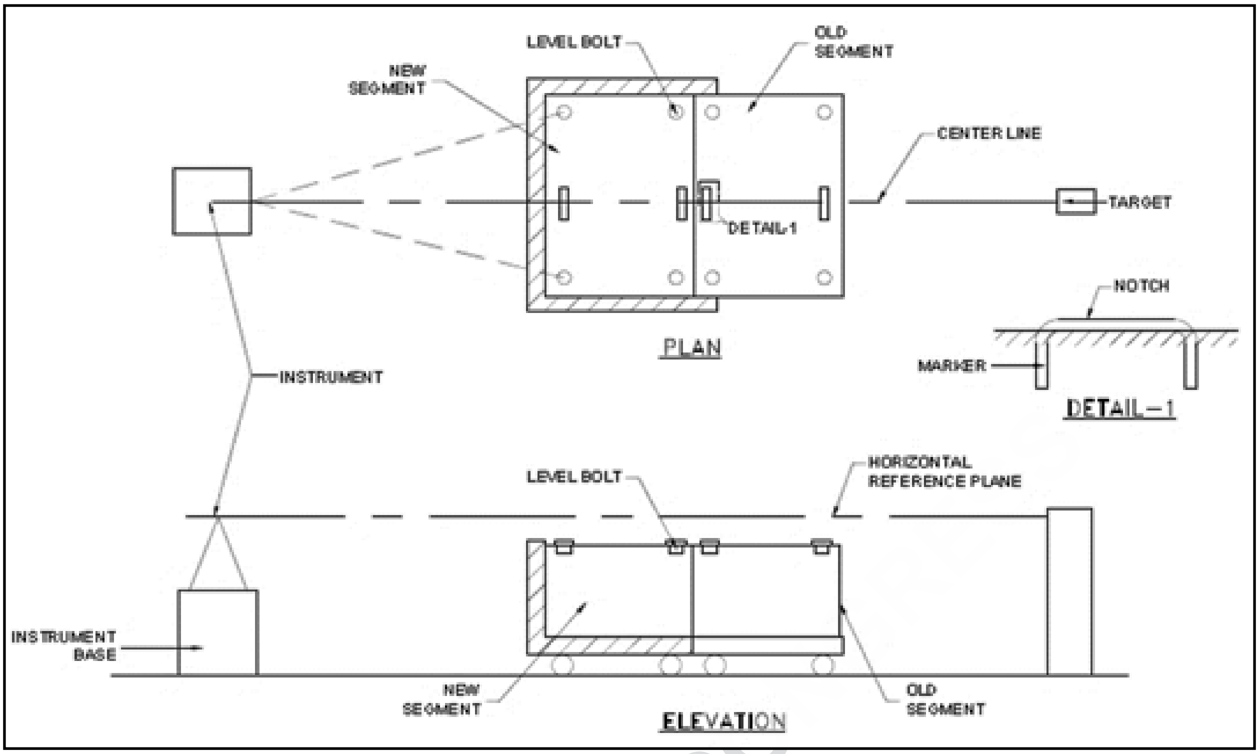


Figure-6.24: Alignment control setup in Short line casting (1) [1]

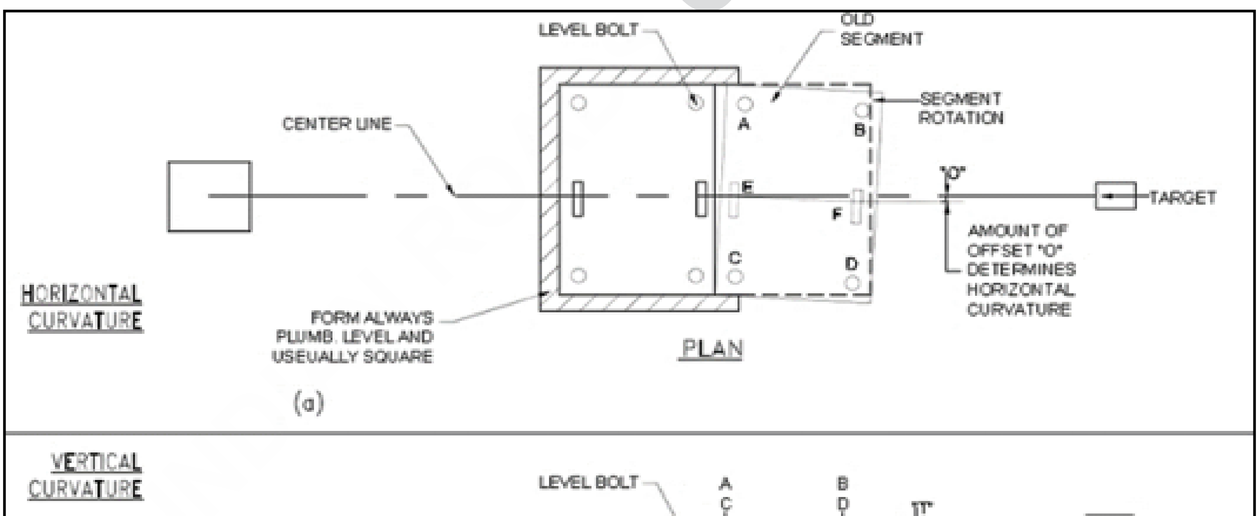


Figure-6.26: Derrick Crane erection arrangement [9]

6.4.3 *Construction Tolerance of Balanced Cantilever Construction*

During erection by balance cantilever method, the following tolerances should be adhered to:

Maximum differential displacement between outside faces of adjacent segments in erected position should not exceed 5mm.

Angular deviation from theoretical slope difference between two segments, should not exceed 0.001 radians in transverse direction and 0.003 radians in longitudinal direction.

Dimensions from segment to segment will compensate for any deviations within a single segment so that the overall dimensions of the completed structure meet the dimensions shown in the Plans such that the accumulated maximum error does not exceed 1/1000 of the span length for either vertical profile and/or horizontal alignment.

Erection of first segment, after construction of pier table is very critical in geometry control and enough caution should be given during the erection. This first segment recommended to be placed within an accuracy of 0.3 mm.

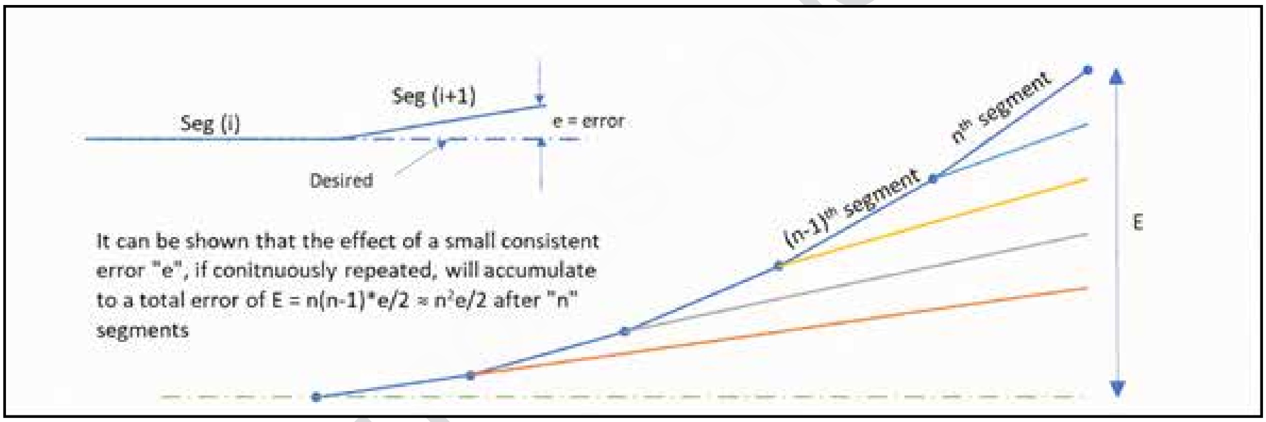


Figure-6.27: Effect of Accumulation of Systematic Errors during erection of pier table segment [1]

The geometric control plan should be prepared which describes the detail how the survey is to be performed and how to assure proper erection of the structure to the final grade as shown on the drawings. The geometric control plan should provide for regular monitoring of the superstructure deflections beginning with the addition of the first cantilever segment and concluding with the last cantilever segment. The plan should include the adjusting procedure to be utilized, should the cantilever, as erected, deviate from the predicted alignment by more than 1.0 in. (25 mm).

The elevations and alignment of the structure should be checked at every stage of construction and must maintain a record of all these checks and all adjustments and corrections made. All surveying should be performed at a time that will minimize the influence of temperature. Corrections by shimming should be done only when approved by the Engineer. For precast segmental construction using short line forming techniques, precision surveying systems should be used so that levels and horizontal alignment during precasting are measured with in an accuracy of ±0.01 in. (±0.3 mm).

For precast segmental construction using match-cast segments, careful checks of both measurements and computations of geometry should be made before moving segments from their casting position. Theoretical coordinates of all sections cast should be completed before casting a new segment. In addition to the computed and as-built casting curves for vertical and horizontal deflections, a cumulative twist curve should be computed using the measured cross slopes of the individual units as a check on the extrapolated deflections. In computing set-up elevations in the match-cast process, priority should be given to correcting twist errors by proper counter-rotation. The segment in the match-cast position should not be subjected to a stress inducing twist.

Figure-6.28 (a): Shimming Joints to Correct a Profile during erection [1]

If observed profile does not match desired profile and if it is clear that they will continue to diverge, then some corrective action is possible by shimming the joints with approved material. Usually the remaining cantilever segments have already been cast by this time making corrections by casting compensations impossible.

Shimming should be done only if absolutely necessary and then it should be kept to a minimum because this effect is not predictable and the thicker joints can lead more intrusions of epoxy into ducts, etc.

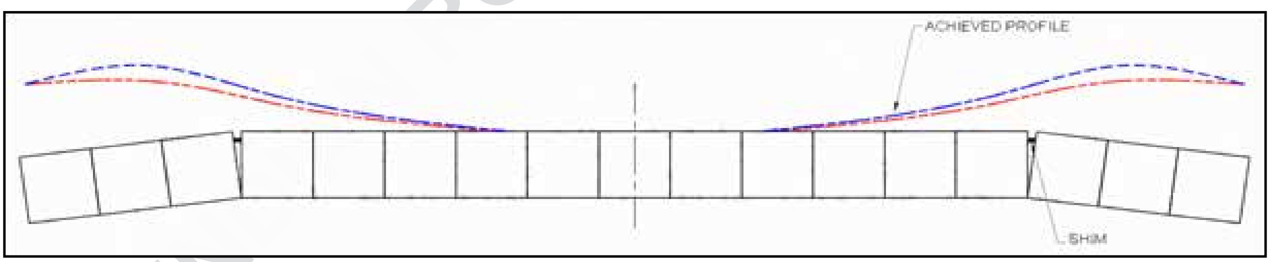
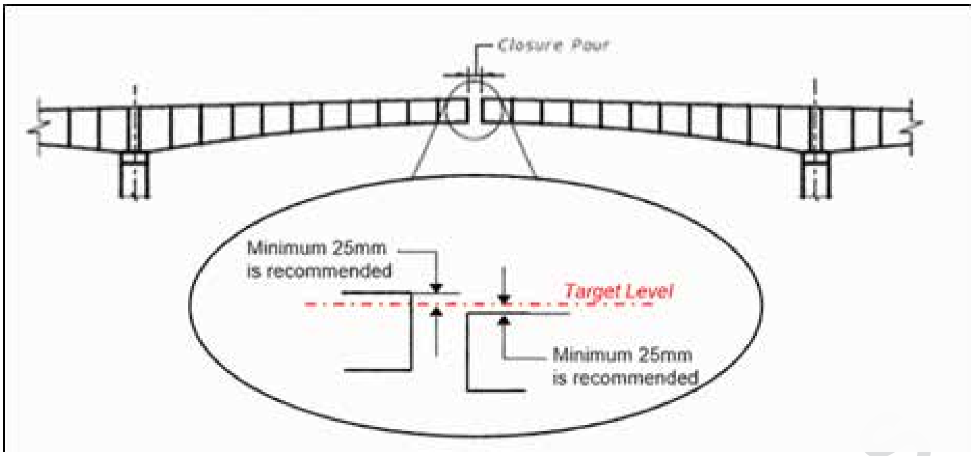


Figure-6.28 (b): Shimming Joints to Correct a Profile during erection [1]

When cantilever construction is used, the tolerances for the alignment of the opposing cantilevers in a span should be stated in the contract documents. The forces up and down which can be placed on the end of the cantilever should also be stated on the drawings and should consider the allowable tensile stresses for construction load combinations permitted by the code. (Refer Figure 6.29)



Figure–6.29: Example of Construction Tolerance at both end of cantilever span before closure [3]

Cast-in-place closure segment can be considered in order to mitigate easily the relative construction tolerance of both cantilever tips and accommodate the continuity tendons within the closure joint.

6.5 Case Studies

Extradosed bridges can be constructed with different span arrangements, Deck, Pylon and Cable System based on functional, design and construction limitations. 3 different case studies discussed in this chapter which are having their own unique criteria.

A. 3rd Narmada Bridge, Bharuch – Multicell PSC Box Girder with Twin Pylon & Cable System.

B. Durgam Cheruvu, Hyderabad – Fish belly shape PSC Box Girder with Central Pylon & Cable System

C. New Ganga Bridge, Patna – Inner & Outer Struts PSC Box Girder with Central pylon & Cable System

6.5.1 *3rd Narmada Bridge, Bharuch*

Narmada bridge is India's longest extradosed cable stay bridge which was opened to traffic on March 2017.The length of the main bridge is 1.34km with two end spans of 96m and nine internal spans of 144m. The superstructure consists of 20.8m wide precast segmental concrete box girder, with a carriageway to accomodate four lanes of traffic. Narmada bridge design and construction comprises of bored cast-in-place pile foundation, pier-cap, Y-shaped pylons, and extradosedcable deck which consist of 3.55m length segments having three-cell precast segmental box girder castwith short-line method and erection by balance-cantilever method using by Bridge Builder. Stay cables and post tensioning systems were provided to have structurally sound decking system.

Figure-6.30: Narmada Bridge, Bharuch [9]

The bridge has segmental precast girders erected in balance cantilever manner on both sides of pylons supported on pile foundation. Concept was more or less module based design. The bridge consists of nine pylons having balance cantilever extradosed super structure and two abutments . Bored cast-in-place piles of 1.5m diameter have been provided with over 2.25m thick pile-cap. Top of pile cap is at low water level. Vertical pile capacity is duly modified for scour condition by considering overburden pressure from scour level.The typical pylon for the main bridge substructure was chosen to have a Y-shaped with rounded corners to improve aesthetics and to reduce wind and water current loads. To cast this kind of shape, special steel formwork with adequate scaffolding and false-work system was designed and provided, to have control over geometry of structure.

The super-structure of the main bridge consists of three-cell of precast segmental box girder having depth of 4.0m. Sloping outer webs connects the top slab and the inner vertical webs to stabilize the top slab in transverse direction and to transfer stay force to the bottom of inner/vertical webs. Soffit corners are rounded due to presence of transverse tendons in outer sloping webs and to reduce drag coefficients under wind loadings. The length of typical segment was limited to 3.55m in order to restrict the weight of the segments during handling. Segments were to be match-cast. Integral connection at the pylon location of substructure and the superstructure was provided by pier tables. Anchor saddle boxes were provided at upper pylon which provides individual support for each strand and avoid lateral pressures due to grouping of strands. Balanced cantilever construction method was used to erect the box girders with epoxy joints between segments. For service and ultimate load condition adequate internal post-tensioning is provided.

Construction of the bridge superstructure involved different phases shown schematically in Figure 6.31.

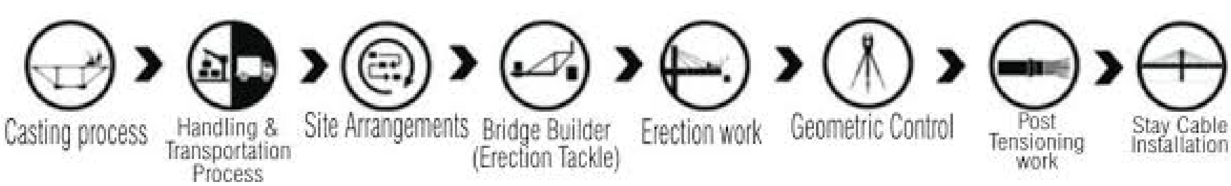


Figure-6.31: Bridge Span Constriction phases

The precast segmental Bridge Builder (Figure 32) was designed for a maximum precast segment length of approx. 3 - 5 m and load capacities between 100 tons and 300 tons. It was equipped with two hoists for lifting the precast segments as well as for adjusting the cross-fall. A manipulator permitted the adjustment of the longitudinal fall and hydraulic cylinders launch the device forward. The Precast Segmental Bridge Builder was designed and adopted for balance cantilever precast segment erection. The bridge builders on both ends of the spans initially were erected on the pier-table only. Then one by one precast segments were lifted simultaneously and fixed as mentioned in the erection scheme involving prestressing and stay cable system.

Figure-6.32: Bridge Builder arrangement on Pylon-span [9]

The erection procedure included 5-cyclic activities, which were listed in Figure 6.33. Erection method was precast-segmental balance cantilever erection system..

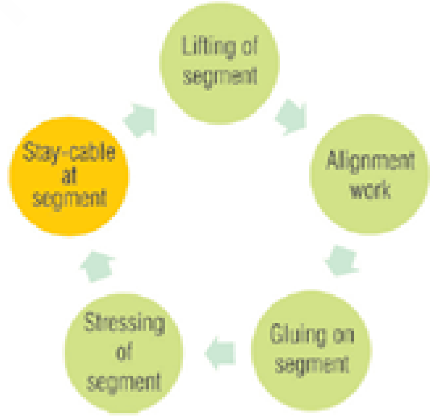


Figure-6.33: Erection work flow

6.5.2 *Durgam Cheruvu, Hyderabad*

6.5.2.1 *Introduction*

Greater Hyderabad Municipal Corporation, Hyderabad (GHMC) under the aegis of Municipal Administration & Urban Development Department (M.A. & U.D) Government of

Telangana (GOT) has taken up steps for the development of infrastructure in Hyderabad. As a part of this development, GHMC has proposed the "Construction of Extradosed Bridge across Durgam Cheruvu Lake at Madhapur in Hyderabad under Engineering Procurement and Construction (EPC) Turnkey".

Figure-6.34: Location of Project [9]

6.5.2.2 *Geometry*

The main span of Extradosed Bridge over Durgam Cheruvu Lake is 233.85m. In terms of main span of Extradosed Bridge, this is the longest bridge in India and within top 10 bridges of the world.

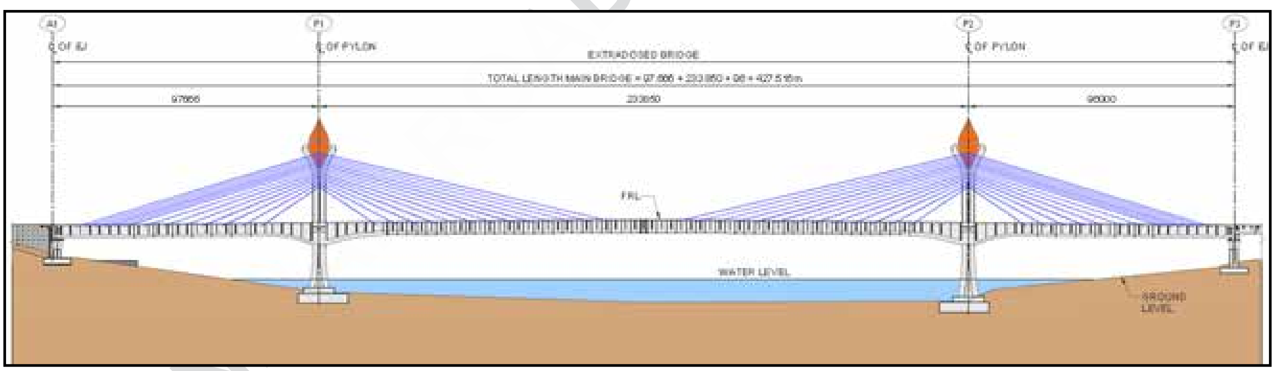


Figure-6.35: Elevation and Span Arrangement [9]

The bridge is a three-span continuous with span arrangement of 97.66m+ 233.85m+96m, deck is connected monolithically at two pylons and supported over bearings at the end piers at Expansion Joint location. Lower pylons are at different heights due to different bed levels. Height of lower pylon P1 is of 15.78m and P2 is of 19.0m. Open foundation is adopted due to availability of weathered rock at a shallow depth. Client proposed single plane stay cables in the median. Part of the load from superstructure is transferred to upper pylons by 13 no's stay cables on each side of pylon.

In general, Extradosed bridges have symmetric shape and symmetrical cable arrangement about the pylon. Generally, the cantilever erection method is adopted and the end part of the back span will be cast-in-place/ precast erected on staging from the ground. However, this bridge is planned asymmetric with technical challenges such as controlling the girder stress at mid-span and uplift at the back span.

Depth of box girder is 4.5m and varies up to 7.5m near to the pylon over a length of 25m on both sides of the pylon. As the deck width of the superstructure is 25.5m and the bridge is in seismic zone II, the weight of the superstructure deck is minimized by providing ribs and internal struts. Internal struts at cable anchorage zone is strengthened by internal tendons to transfer the stay force to webs. Typical cross-section of box girder with ribs and internal struts are shown in Figure 6.36(a) & 6.36(b).

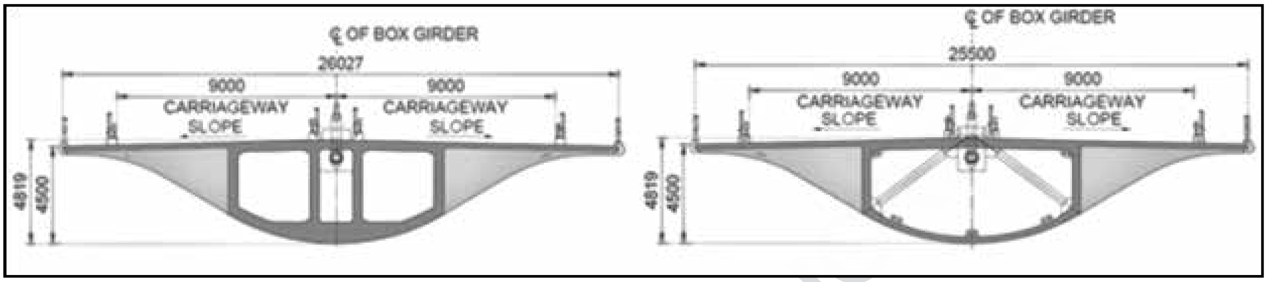


Figure-6.36(a) Figure-6.36(a)
Typical Cross Section at Back Span Side [9] Typical Cross Section at Main Span Side [9]

Based on field and laboratory test results, it was found sub-surface profile generally consisted of completely weathered rock underlain by granite bedrock. Hence Open foundation with 17m(W) x 18m(L) was adopted for the pylon. Center of foundation was shifted to 2.0m (towards main span) from the center of pylon along the traffic direction to nullify the residual bending moment induced by unbalanced permanent loading at final construction stage. General arrangement of foundation and pylon is shown in Figure 6.37.

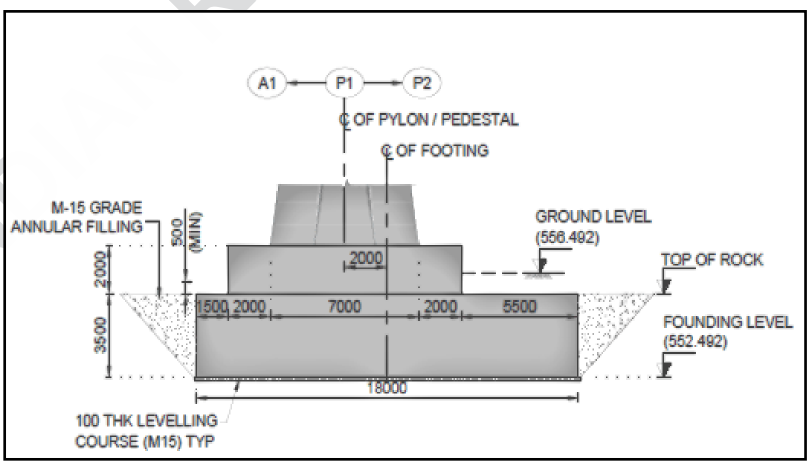


Figure-6.37: Typical Cross Section of Foundation [9]

A study on the connection between the deck and the pylon had been carried out comparing the bearing connection and monolithic connection which restrains partially the rotation of the girder at the pylon location. It was concluded that a monolithic connection without bearing has the merits of reducing the deflection of the main span as well as reducing the uplift force at the end pier location. Although the moment induced at lower pylon due to thermal movement and creep/shrinkage was noticeable range, a monolithic connection at the pylon was adopted as a connection option

In addition, the monolithic connection was also preferred as it provides maintenance-free in terms of bearing replacement which would be a potential risk unless sophisticated care such as restricting the movement of the girder during the replacement is not taken.

To ensure the lower pylon has enough structural strength as well as the capacity to accommodate the thermal displacement and creep shrinkage, a detailed analysis on the pylon had been carried out in order to assess the behaviour against the longitudinal displacement. Based on the assessment of the effect due to thermal movement and creep/shrinkage, it was concluded that the force induced by long term displacement had to be reduced otherwise it was found difficult to find a suitable cross-section. Pre-adjustment of long-term displacement was adopted by using pushback before installing the key segment. Finally, it was concluded that a 233.85m main span with a monolithic connection without bearing was found feasible which also reduces the uplift force at end pier. Size of lower pylon was varying in both directions, along the height. The size of pylon at the top is 4.5mx3.8m and at the bottom it is 7.0mx5.0m

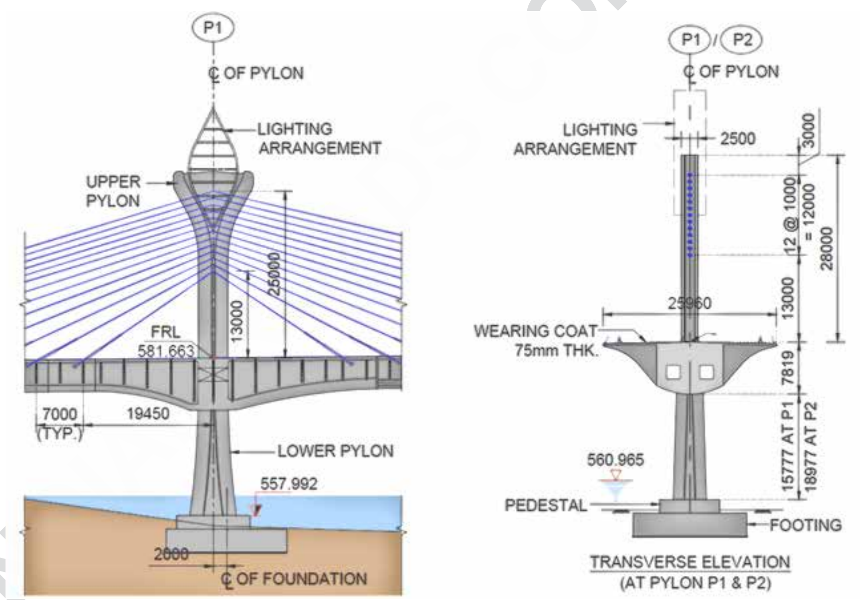


Figure-6.38: Elevation of pylon [9]

6.5.2.3 *Design*

Design concept:

As shown in the Figure 6.35, three span continuous PSC box girder Extradosed Bridge with main span of 233.85m and side spans of 97.6m/96m each was proposed. Because of a comparably shorter back span to the main span length and unbalanced dead load during the erection, balance cantilever erection was ruled out from the construction scheme. In addition, there was a possibility of uplift under anchor piers in permanent loading condition itself. Due to the asymmetric span arrangement of this order, control of the pylon bending moment during construction stage and uplift at the end-pier in-service state were challenging issues in design. To overcome above mentioned difficulties, it was decided to make the section in the back span much stiffer than the main span section. Consequently, weight of the back span increased and uplift reaction at end-pier was eliminated. Hence, construction of back span was proposed to be erected on ground-supported staging and the main span was erected by the cantilever method.

The connection between superstructure and substructure was made monolithic to overcome stability issues during construction and to reduce lifetime maintenance cost. Stay cables were anchored in the deck in the median as per Client's requirement. There were 13 stay cables on each side of the pylon. Due to the stiffness of the deck and less height of the upper pylons, horizontal component of stay forces was found to be higher compared to the vertical component.

Aesthetical aspect:

The client had proposed fish belly shape box girder consisting of the rib as a mandatory requirement in order to improve the architectural view of the bridge. Architectural LED lighting provided under the deck toward the ribs, enhance the night view of the bridge over the lake. The asymmetric single-plane stay cables sustained by the iconic shape pylon were proposed together with architectural lighting on top of the pylon. And this iconic bridge will be a beautiful landmark over the Lake Durgam Cheruvu.

Analysis and Design:

Minimum concrete strength requirements for various components of the structure were as follows as per tender specifications/design requirements satisfying IRC provisions.

Young's modulus of Elasticity for normal-weight concrete is considered as per Table 6.5 of IRC: 112-2011 and the coefficient of thermal expansion for normal concrete is considered as 1.2×10^{-5}/°C.

Table 1: Material Properties	
Element	Minimum Grade
Superstructure for Main bridge	M60
Crash Barrier	M40
Upper Pylon/ Lower pylon	M60
Foundation	M40
PCC	M15
Operating speed for Geometry Design	35 kmph
Design of structural Components	80 kmph

The grade of reinforcement was Fe 500 confirming to IS: 1786-2000. The modulus of elasticity (E) taken equal to 200,000MPa.

The design of the extradosed bridge was developed in accordance with IRC bridge design specifications and PTI recommendations for stay cable design, testing, and installation.

80 kmph design speed was adopted for the project. Further, construction stage analysis was carried out considering the erection sequence, time-dependent material property, and changes in boundary conditions. Also detailed FEM analysis using plate model was carried out in order that the load-transferring mechanism from the Stay cable to the deck and to the internal strut was clarified.

6.5.2.4 *Construction and Erection*

Foundation: To avoid early thermal cracks due to heat of hydration and to maintain the difference in between core and surface temperatures less than 20^{0}C, the self-curing concept by covering the foundation with a polythene sheet and sand filling over the sheet was adopted as shown in Figure 6.39.

Figure-6.39: Construction of Foundation [9]

Lower pylon:

Figure-6.40: Construction of Lower pylon [9]

Pier table:

Figure-6.41: Construction of Pier table [9]

Upper pylon:

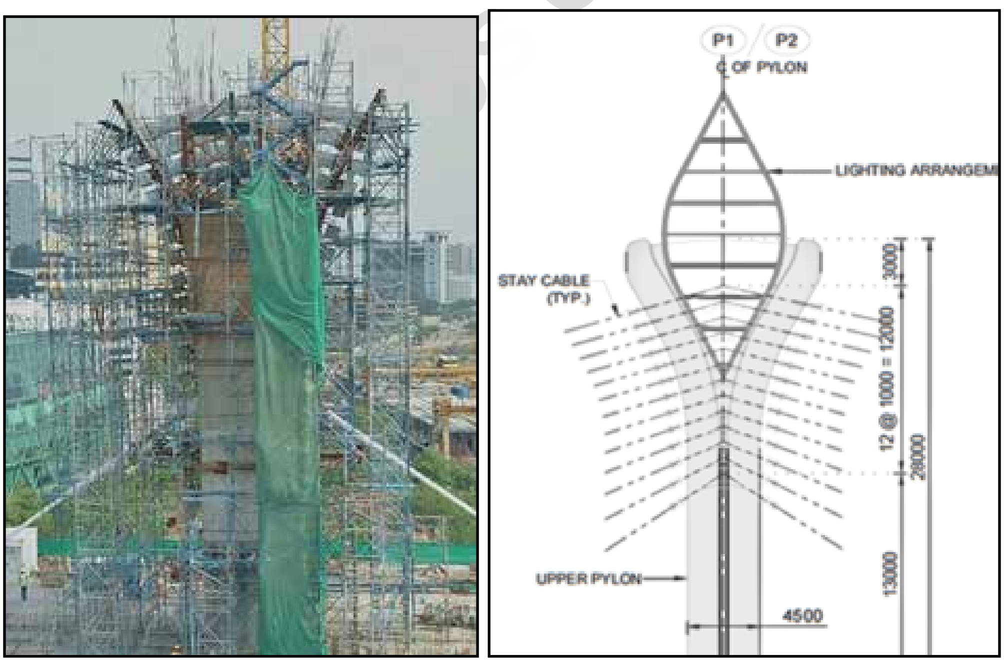


Figure-6.42: Construction of Upper pylon [9]

Deck back span on staging:

Figure-6.43: Construction of Deck back span [9]

Casting yard:

Figure-6.44: Casting yard for precast segments [9]

Deck main span by free cantilever method:

Figure-6.45: Erection of main span by free cantilever method [9]

Mid span stitch:

Figure-6.46: Connection of main span with precast key segment and stitch [9]

6.5.3 *New Ganga Bridge (Kachchi Dargah), Patna*

6.5.3.1 *Introduction*

Bihar State Road Development Corporation Limited under the Government of Bihar has taken up steps for the development of infrastructure in the State of Bihar. As a part of this development, BSRDCL has proposed the "Construction of Green Field Six Lane Extradosed Cable Bridge over river Ganga near Kachchi Dargah on NH-30 to near Bidupur in District- Vaishali on NH-103 at Patna in the State of Bihar under Engineering Procurement and Construction (EPC)". With the total length of 22.76km, the project mainly consists of the construction of the Extradosed Main Bridge section over River Ganga, ramp bridges, viaduct and embankment portion.

Figure-6.47: Site Location Project corridor stretch [9]

The main bridge is across the south and north channels of river Ganga accounting to 9.759km including the island portion and functional requirements of the bridge deck as per Employer requirements and desirable codal requirements shown in Table2. Extradosed concept has been adopted for the entire 9.759km as an Iconic bridge for Patna and Bihar in terms of Tourism and Business Development. The language Require modification

Table 2: Functional Requirement of the Bridge (6 Lanes with median)

Description	Dimensions	
Carriageway	2 X 10.5	= 21.0 m
Crash Barrier	2 X 0.45	= 0.90 m
Footpath	2 X 1.50	= 3.00 m
Shy distance + Paved shoulders	4 X 0.50 + 2 X 1.5	= 5.00 m
Railing Kerb	2 X 0.30	= 0.60 m
Median (Including 2x0.45 median crash barriers)	2 X 0.45 + 2.00	= 2.90 m
Total Width of Bridge	1.4 **m**	

Geometry

The present live channel in the entire bridge length would be 2.7km only, however, the entire main bridge needs to be considered for navigational clearance as the River Ganga can be meandering in due course as per the past 100 years morphology data of River Ganga. The location has been treated as Nation Waterway 01 (NW01) as per IWAI (Inland Waterway Authority of India). Hence expected barge or cargo movement in the river should be high. Accordingly, class VII waterway barges have been considered as per IRC-06 and the navigational requirements of 100m for horizontal clearance and 10m vertical clearance (below the soffit of the girder) above HFL are provided. As shown in Figure 6.48, a typical block has a span length of 450 m and is defined between two expansion joints (Needle Beams Method) for smooth traffic from the previous block to the next block. The connection between superstructure and substructure is made monolithic to ensure stability during construction and to reduce lifetime maintenance costs. Stay cables are anchored in the deck in the median. There are 7 stay cables on each side of the pylon considered in the Design. Due to the stiffness of the deck and less height of upper pylons, stress variation due to traffic load in stay cable is much lesser than in conventional stay cable bridge thus the concern about fatigue is mitigated.

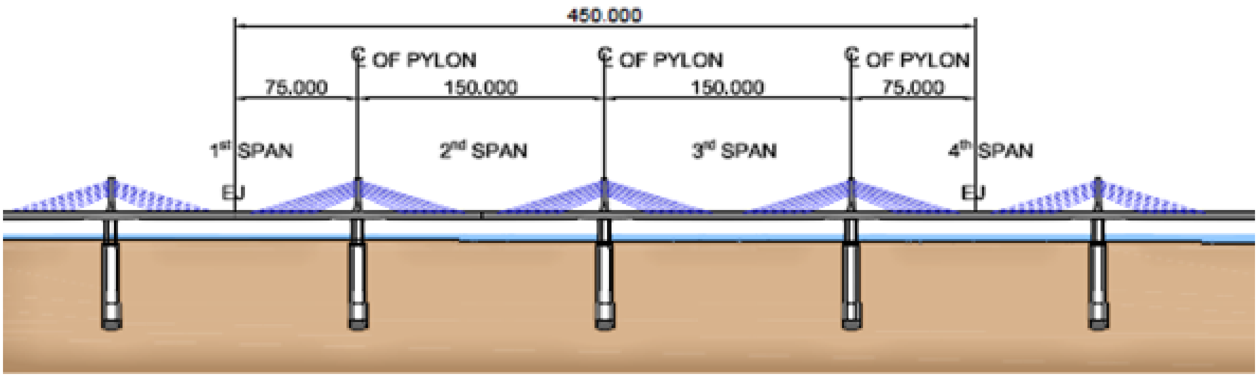


Figure-6.48: Elevation of the Typical Block [9]

Extradosed bridge is proposed in order to improve architectural view of bridge rather than conventional massive girder bridge. The Aesthetic appearance of Kachchi Dargah Bridge is shown in Figure 6.49

Figure-6.49: Aesthetic appearance of Kachchi Dargah Bridge [9]

6.5.3.3 *Structural aspects*

Minimum concrete strength requirements as per design for various components of structure shown in Table 3. Young's modulus of Elasticity for normal weight concrete is considered as per Table 6.5 of IRC: 112 and Coefficient of thermal expansion for normal concrete is considered as $1.2x10^{-5}/°C$.

Table 3: Concrete Grade

Element	Minimum Grade
Superstructure for Main bridge	M55
Crash Barrier	M40
Upper Pylon/ Lower pylon	M55
Well foundation	M35

The grade of reinforcement is Fe 500 and Fe500D confirming to IS: 1786-2000. The modulus of elasticity (E) is taken as equal to 200,000MPa. The design speed adopted in the project is 100kmph for main bridge and 40kmph for loops at the Junctions.

6.5.3.4 *Design Aspects*

The Extradosed Main bridge consists of 22 blocks of two-span continuous with span arrangement of 75m + 2X150m + 75m with three pylons monolithic system and internal needle beam at expansion joints. Lower pylons are of height 18.35m (Typ.). The well foundations are adopted considering scour conditions of the river Ganga. Part of the Span load from the superstructure is transferred to upper pylons by 7 number stay cables on each side of pylons.

The depth of box girder is typically 4.0m and it varies over a length of around 14.5m near the pylon's location. As the deck width of the superstructure is 32.4m and the bridge is located in highly seismic area (Zone IV), in order to minimize the weight of the superstructure deck has been provided with internal and external struts along with Transverse PT for the top slab of the deck. Internal struts at cable anchorage zone are strengthened by internal tendons to transfer the stay force to webs. Typical Cross sectional view of a precast segmental box girder with internal and external struts is shown in Figure 6.50

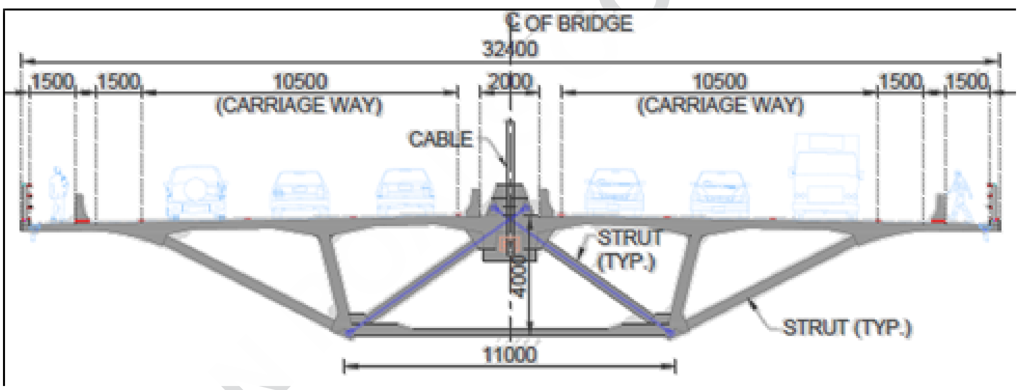


Figure-6.50: Sectional Elevation of Girder [9]

Double leaf reinforced concrete pier is adopted for the lower pylon which is connected to the girder monolithically. This proposed lower pylon shape has key merits as follows. Its flexibility against thermal or long-term displacement provides a feasible solution to eliminate bearing. Obviously the issue of bearing maintenance is not a concern with this arrangement. Expansion joints are located every 450m spacing typically and provide better comfort level to the drivers and also reduce the overall maintenance cost. However in order to mitigate the long terms displacement due to creep/shrinkage, pre-set back of both cantilever tip by using push back before closing the key segment is adopted.

The Scour parameters of the bridge under consideration is as follows below. General arrangement of foundation and pylon section plan are shown in Figure 6.51

a.	High Flood Level (HFL)	50.00m
b.	Low Water Level (LWL)	41.00m
c.	Design Scour Level	03.50m
d.	Design Discharge	106,839cumecs
e.	Maximum Scour Depth	46.5m from HFL

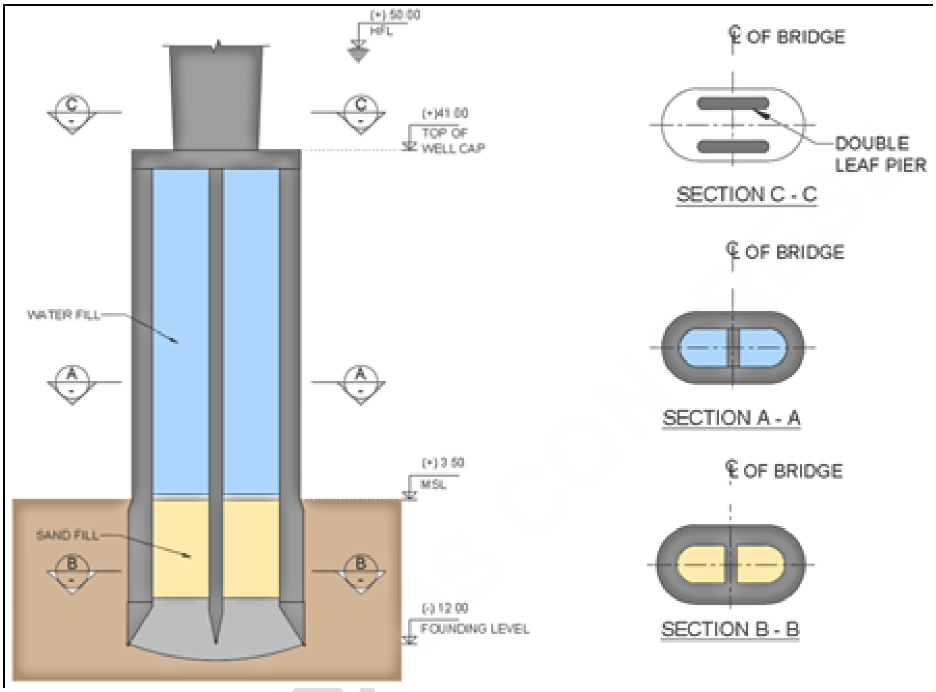


Figure-6.51: Well Foundation Details [9]

6.5.3.5 *Construction and Erection*

<u>Lower pylon:</u>

(i) Lower pylon construction in Water mode *(ii) Lower pylon construction in Land mode*

Figure-6.53: Construction of Lower pylon [9]

Pier table:

(i) Pier Table Shuttering (ii) Pier table concrete section

Figure-6.54: Construction of Pier table [9]

Upper pylon:

Figure-6.55: Upper Pylon Construction [9]

Deck main span by free cantilever method:

Figure-6.56: Erection of main span by free cantilever method [9]

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SECTION 7

LOADS, LOAD COMBINATIONS AND DESIGN OF EXTRADOSED CABLE

Section I: Loads

The description of actions, their classification, the nomenclature of design situations, and the combination for verifying different limit states as explained in IRC 112, are also included in this guideline. The loads and actions on extradosed cables and on account of the extradosed cables are based on the *fib* 89 [1].

7.1 Dead Load

Dead loads should be in conformity with the provision of IRC 6. The unit weights of the material should be in accordance with the provisions of Clause 203 of IRC 6.

Extradosed cable Weight

For extradosed cable, the weight of the main tension element, and any additions to the cables, such as sheaths, grout, etc., should be included as dead load. The Self-weight of the extradosed cable and its connection including sheathing should be as per the recommendations of the stay cable system supplier. In the absence of any data, the following can be used

Weight of parallel wire / parallel strand extradosed cable = A_s x n x g_s

Where

A_s - Nominal area of wire / strand,

g_s - Density of the steel 7850 kg/cum

n – number of wires / Strands

The Weight of the protection material viz. Heavy galvanization, Sheath, and grout can affect the weight of the cable and increase its weight by 10% to 25%. Fig 7.1 shows an example where a 7-strand prefabricated cable (GJ15-7) has a weight 9.79 kg/m (i.e. about 25% additional weight of protection)

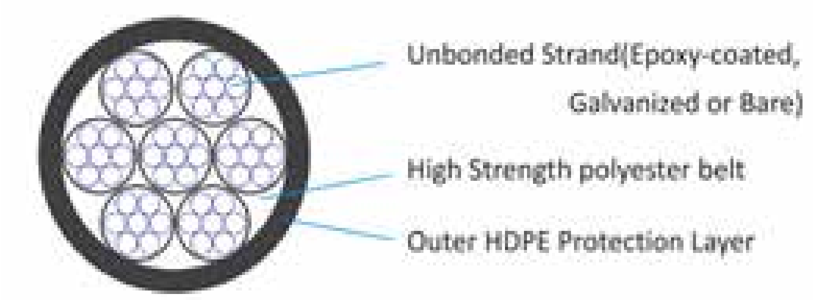


Fig: 7. 1 Prefabricated Cable GJ 15-7

7.2 Prestressing Force in concrete

In analysing the structure Prestress is considered as a force acting on concrete elements which has time dependant variation. Two characteristic values of prestressing force should be used (i,e. g_{sup} (i.e. higher estimate) and g_{inf} (i.e. lower estimate).

7.3 Secondary Effects

In the design of extradosed bridges, the effects of shrinkage and creep of concrete should be considered as per section 6 of IRC 112 and load factors and load combinations should be considered as per IRC 6

7.4 Super Imposed Dead Load

Loads from handrails, crash barriers, bridge furniture, footpath, and actual or provisional loads from utility services, etc. are covered under this loading. Partial safety factors and load combinations should conform to provisions of IRC 6

7.5 urfacing and Wearing Coat

Different partial load factors are to be used depending upon whether the load is adding to the effect of variable loads or relieving. This is to account for re-surfacing, with or without removing the existing coat, and the possibility of change in the type of surfacing. Partial safety factors and load combinations should conform to provisions of IRC 6

7.6 Snow Load

The effects of Snow Load, wherever applicable, should be considered as per provisions of IRC 6

7.7 Vehicular and pedestrian live load (Q)

All the effects of vehicular live load including vertical load effects, impact factor due to vehicular live load, longitudinal forces, centrifugal force and pedestrian live load should be considered as per the relevant clauses of IRC 6

7.8 Fatigue Load

The performance of the extradosed cable is strongly influenced by its fatigue performance under repeated live load cycles. For all concrete/ steel components of the extradosed bridge, excluding the extradosed cables and its connections, provision of IRC:6 for fatigue load and relevant annexure of IRC112, provision of IRC:22 and IRC:24 should be applicable. For stay cable design, provisions given herein should be followed. The fatigue performance of extradosed cable will depend on the stress range, which is the magnitude of the live load stress in cables as a proportion of the dead load. All extradosed bridges need to be designed for fatigue limit state as explained in the design section of this chapter.

Fatigue Truck to be considered should be in accordance with Clause no 204.6 of IRC 6. The stress range resulting from the single passage of the Fatigue load along the longitudinal direction of the bridge should be used for the fatigue assessment with the fatigue load so positioned as to have the worst effect on the detail or element of the bridge under consideration. The Fatigue truck should be placed on the carriageway with respect to the kerb accordance with IRC 6. In the case of a divided carriageway, a single passage of fatigue load should be considered for each carriageway.

7.9 Thermal Loads

All steel and concrete bridges should be designed for IRC 6 requirements for uniform daily and seasonal temperature ranges and thermal gradient effects. In addition, non-uniform temperature variations should be considered. Preference should be given to using a light - coloured HDPE pipe.

A temperature difference between the HDPE or other sheathing and the MTE (Main Tension Element) may affect the design of any expansion joint details for the PE pipe.

Following Thermal effects should be considered for design.

(i) Uniform Temperature rise/fall as per IRC 6

(ii) Thermal gradient through the deck (Positive and Reversal Gradient as per IRC 6)

(iii) Differential temperature between various members of the structure, especially between the cables and the rest of the bridge, must be considered in the design.

For temperature difference between cables and deck or between cables and pylon, values of 10°C and 20°C should be used for brightly coloured HDPE pipe, and black HDPE pipe, respectively. Similarly, 5°C linear temperature difference between the opposite faces of pylon should be also considered.

All the above effects should be considered simultaneously or independently whichever is unfavourable for SLS checks. For ULS checks gradient effects across the section should be neglected.

7.10 Cable replacement

The design of extradosed structures should provide for the replacement of any individual extradosed cable with a reduction of the live load in the area of the cable under replacement. The closure of the traffic lanes nearest to the cable should be considered for the design.

Some cable systems offer possibility of replacing cable strand by strand. In such situations replacement method should be clearly indicated in the Maintenance Manual.

7.11 Cable Breakage

All extradosed bridges should be capable of withstanding the accidental loss of any one cable without the occurrence of structural instability.

According to SETRA [2], The impact dynamic force resulting from the sudden rupture of a cable should be 1.5 (dynamic amplification factor) times the static force in the cable.

DAF of 1.5 is recommended for parallel strand cables and parallel wire cables. This force should be applied at both the top and bottom anchorage locations in the direction opposite to the cable tension.

Cable breakage should be considered as Accidental load for design verification. Forces in other cables under this situation should be limited to 0.8 $F_{GUTS.}$

7.12 Transverse Loads applied from the cables to the structure.

Based on *fib* 89 [1], Guiding devices installed near the cable anchorages or saddle exits laterally support and limit the transverse displacements of cables at this location. As a consequence, these devices protect the cable anchorages from the effects of transverse loads and the tensile elements from excessive angular deviation a_{SLS}.

The transverse support provided by the guiding device to the cable causes a kink in the cable geometry, see Fig. 7.2, and a corresponding transverse load is applied by the cable to the guiding device as well as the structure supporting the guiding device.

The structure supporting the guiding device needs to be designed for the maximum transverse forces, F, applied from the cable. These transverse forces are the product of angular kink and axial cable force T_{SLS}, usually the maximum force during construction and service limit states.

For detailed design angular deviations should be considered from the following,

- Deformations of the structure and change of cable sag due to construction loads, wind and traffic loads, and due to temperature changes in the service condition
- Cable vibrations
- Construction tolerances

For the preliminary design of the structure supporting the guiding device installed in a cable, an angular kink of $\alpha = \pm 1.4$ degrees (± 25 mrad) is suggested as a reasonable assumption for SLS. Careful detailing for the connection of the guide pipe and its extension is required to allow the connection of both pipes considering the construction tolerances for the alignment of the guide pipe and the actual catenary profile.

For systems without a guiding device, care should be taken that the cable will not touch the guide pipe at excessive angular deviations, including construction tolerances, and that the anchorages are designed to resist the maximum transverse load as mentioned above. The lateral load should be applied to the cable anchorage at the relevant location.

Also, other transverse loads, such as the weight of cable pipe including vandalism protection and loads from internal dampers need to be considered.

The transverse force can be computed as follows:

7.12.1 *Ultimate Limit State*

Where,

- Load Factor 1.5
- Max Tension in cable under Rare Combination

7.12.2 *Serviceability Limit State*

No service criteria are specified unless the guide pipe constitutes a support for a damper, in which case the maximum deflection of the guide pipe under SLS should be defined by the damper supplier.

7.12.3 *Fatigue Limit State*

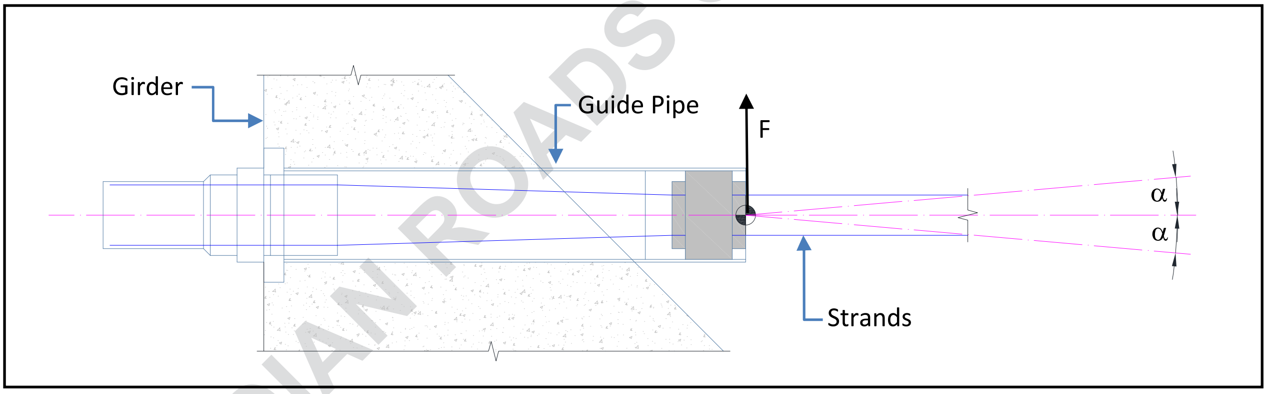
The following can be considered:

Where,

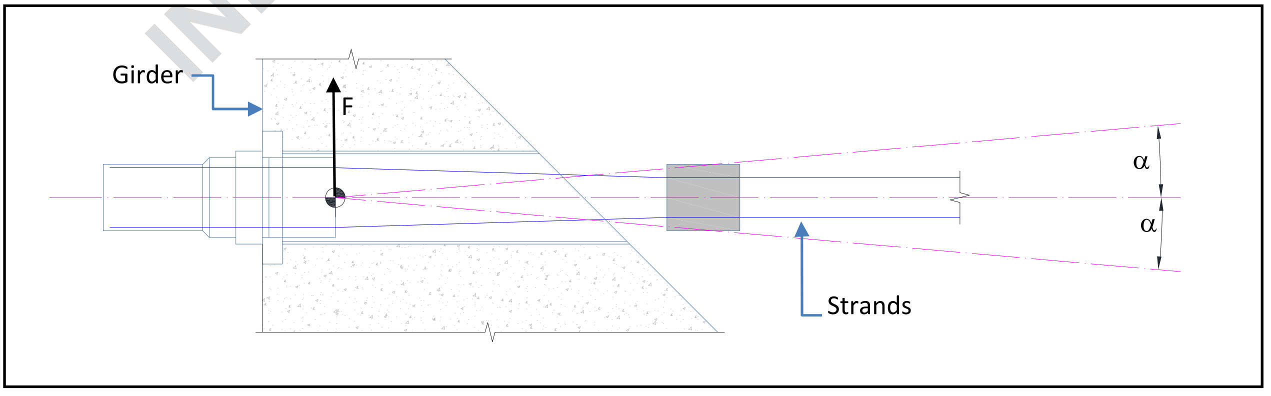
F_{perm} Tension in cable corresponding to the service loads under frequent load combination.

$D\alpha_{FLS}$ corresponds to ± 10 mrad

When the guide pipe constitutes only a support for a damper this value may be reduced according to the type of damper.



a) Elevation of anchorage system with guiding device



b) Elevation of anchorage system without guiding device

Fig 7.2: Transverse force by cable angular deviation

7.13 Bending stresses in cables

fib 89[1] provides guidelines for methods for considering bending stresses in cable, depending on the actual cable force and cable weight, and in the absence of other loads, the cable will take a specific catenary cable profile. The cable will be subjected to pure axial tension only if the anchorages at either cable end are placed along the tangent to that specific catenary cable profile. Any deviation of the actual cable profile from the above, due to:

- transverse displacement of the cable under loads applied to the cable or structure
- rotation of the anchorages relative to the tangent because of applied loads, temperature changes, or installation tolerances of anchorages, bearing plates, and guide pipes
- cable vibrations
- change of sag of the cable due to change of cable force

will introduce local bending stresses into the cable, in the region where the transverse displacement or rotation is applied. Devices such as cable anchorages installed with an inevitable placement tolerance, guiding devices, saddles, and clamping devices along the cable length, e.g. for cross ties, introduce such deformations and corresponding bending stresses.

The cable rotations at the anchorage and saddle exit should be calculated by the designer. When using extradosed cable systems designed and tested in accordance with these recommendations, fatigue angular deviations and service angular deviations applied at the anchorage or saddle up to ± 0.6° (± 10 mrad) with 2 million cycles are covered by the system design and testing and do not need to be considered in the design. Consequently, it is not required to add bending stresses to axial stresses.

For structures with larger angular rotations, the designer has to specify the usage of a guiding device or specify suitability testing, including fatigue, service, and strength, with the larger rotations.

7.14 Fire, impact, vandalism

Design for fire protection if required should be based on qualification testing of fire-resistant cable systems which protect cable from reaching a temperature greater than 300 degrees C for a period of not less than 30 minutes. The Fire Condition should be considered an Accidental Event with cable dynamic force = 0.0 (Cable Loss from fire is assumed to be gradual).

7.15 Wind Loads

Static wind actions as defined in IRC 6 and annexure A4 should be considered for the design of extradosed bridges. Dynamic effects of the wind on cables, should be accounted as per annexure A4.

7.16 Seismic Loads

The seismic design of extradosed bridges should be carried out as per the provisions of IRC SP: 114. Analysis should be carried out based on chapter 9 and Annexure A6 of this document.

7.17 Cable forces

The cable forces are of two types i.e.,

1) cable force at the end of the staged construction which includes dead load as well as cable shortening.

2) force due to tension from the various actions on the structure viz. other dead loads, superimposed load, live load, thermal, etc.

The tension in stay due to other various actions as mentioned in (2) above on the constructed structure should be arrived at directly from the analysis of each action.

7.18 Loads during Construction.

The construction loads to be considered for design purposes should be in line with the approved construction scheme drawings for the bridge. The effect of locked-in stresses due to erection loads should be accounted for.

Checks should be made to evaluate the effects of creep of the superstructure concrete on change of force in the extradosed cables and related changes in stresses in the superstructure during construction.

For design during construction stages, clauses under Annexure A6 of IRC 112 should be referred.

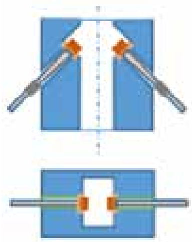
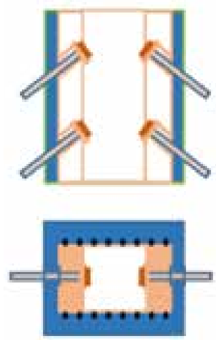
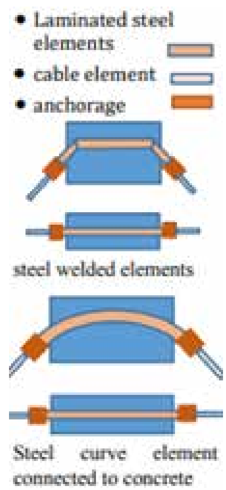
7.19 Saddles

Saddles that allow cables/MTE to pass through them instead of anchoring at a pylon may be used with the following checks:

1) If Saddles which allow cables/MTE (Type D of Fig 7.3) to pass through them are used, the variable stress in the cable due to frequent live load should have to be limited to 50MPa. This check is not required for an Anchor box type saddle (Type A, B & C of Fig 7.3) with anchorage outside the pylon.

2) Saddles must be designed to ensure a safe transfer of vertical forces and differential forces of cables from opposite sides of the pylon (main span and side span) into the pylon structure in the erection and final state.

<table>
<tr><th>Type</th><th>A</th><th>B</th><th>C</th><th>D</th></tr>
<tr>
<td>Elevation

Plan</td>
<td></td>
<td></td>
<td>
</td>
<td>

Courtesy-M/S Freyssinet International


Courtesy-M/s VSL</td>
</tr>
<tr>
<td>Comments</td>
<td>Cables anchored inside concrete pylon</td>
<td>Cables anchored inside composite element or in

Mast head steel</td>
<td>Cables anchored in cast-in steel element</td>
<td>Individual holes for individual strands of cables passing over saddle in concrete pylon</td>
</tr>
<tr><td colspan="5">Fig 7.3 Anchorage configuration in pylon</td></tr>
</table>

7.19.1 *Transfer of differential forces by friction.*

For friction saddles, either a bundle of tensile elements in a pipe or each individual tensile element in a single tube, the maximum differential force should be evaluated with the following expression:

$$\max \frac{T_{sup}}{T_{inf}} < e^{\frac{\mu_{eff}}{\gamma_{M,fr}}\theta}$$

Where,

- T_{sup} (resp. T_{inf}) is the highest (resp. lowest) between the respective concomitant cable tensions on both sides of the saddle at the relevant verification state
- θ is the saddle angle in radians.
- μ_{eff} corresponds to the effective friction coefficient between the tensile

elements and the saddle. The effective friction coefficient should be taken as the minimum of the two values obtained from the two friction tests.

- $\gamma_{M,fr}$ is the material factor to be applied to the effective friction coefficient and should be not less than 1.5

Slippage should be avoided at any state of the structure including ULS.

Section II: Load Combinations and Load Factors

For the design of the elements of the extradosed bridges (viz, Deck, Pylon, Pier Foundations) load combinations as per IRC 6 for Stability, Serviceability and Strength should be followed. Additional combinations in respect of Extradosed cables as given below should be considered.

7.20 Combinations for SLS check

Table: 7.1 Partial Safety factor for verification of Serviceability Limit State for extra-dosed cables

Loads	Rare Comb 1	Rare Comb 2 (Congestion factor)	Rare Comb 3 (SV Loading)	Cable Replacement
Permanent load:				
Dead Load,	1.0	1.0	1.0	1.0
Creep and Shrinkage effects	1.0	1.0	1.0	1.0
Earth Pressure due to Back fill	1.0	1.0	1.0	1.0
Surfacing:				
a) i) Adding to effect of variable loads	1.2	1.2	1.2	1.2
ii) Relieving the effect of variable loads	1.0	1.0	1.0	1.0
Settlement Effects:				
a) i)Adding to Permanent loads	1.0	1.0	1.0	1.0
b) ii) Opposing to Permanent loads	0	0	0	0
Variable Loads:				

Live Load		*		**
a) i)Leading Load	1.0	1.0	-	1.0
b) ii)Accompanying Load	0.75	0.75	-	0.75
iii)with Congestion factor	No	Yes	No	No
c) *iv) Special Vehicle Loading*	-	-	1.0	-
d) v) Braking	1.0	-	-	1.0
e) vi) Centrifugal	1.0	-	-	1.0
Wind Loads				
a) i)Leading Load	1.0	1.0	-	-
b) ii)Accompanying Load	0.6	0.6	-	-
Live Load surcharge	0.8	0.8	-	-
Thermal Loads				
a) i)Leading Load	1.0	-	-	-
b) ii)Accompanying Load	0.6	-	-	-
Cable Load	1.0	1.0	1.0	1.0

* Live Load without transverse eccentricity ** Reduced Live Loads Lanes

7.21 Combinations for ULS check

Table: 7.2, Partial Safety factor for additional verification of extradosed cable strength

Loads	Basic combination	Basic combination (Congestion factor)	Basic combination (SV Loading)	Cable Break (Accidental)
Permanent load: Dead Load, SIDL except surfacing a) Adding to effect of variable loads b) Relieving the effect of variable loads	1.35 1.0	1.35 1.0	1.35 1.0	1.0 1.0
Surfacing: b) i)Adding to effect of variable loads c) ii)Relieving the effect of variable loads	1.75 1.0	1.75 1.0	1.75 1.0	1.0 1.0

Earth Pressure due to Back fill: a) i)Leading Load b) ii)Accompanying Load	 1.50 1.0	 1.50 1.0	 1.50 1.0	 1.0 1.0
Variable Loads:				
a) **Live Load** b) Leading Load c) Accompanying Load d) Construction live load *with Congestion factor* e) *Special Vehicle Loading* f) Braking g) Centrifugal	 1.5 1.15 1.35 No - 1.5/1.15 1.5/1.15	 1.5 1.15 1.35 Yes - - -	 - - - No 1.15 - -	 0.75 - - No - 0.75 0.75
Wind Loads a) i)Leading Load b) ii)Accompanying Load	 1.50 0.9	 1.50 0.9	 - -	 - -
Thermal Loads c) i)Leading Load ii)Accompanying Load	 1.5 0.9	 - -	 - -	 - -
c) Live Load surcharge	1.2	-	-	-
d) Erection load	1.35	-	-	-
Prestress in extradosed cable	1.0	1.0	1.0	1.0
Accidental Effects: a) Cable break	 -	 -	 -	 1.1

7.21.1 *ULS Check – Standard Method*

As per this approach for the design of superstructure, cables, and pylon, prestress in extradosed cable should be factored by 1.00, with g_p =1.00,

7.1.2 *ULS check – Alternative Method*

The standard method as explained above is conservative. An alternative approach for ULS check for extradosed bridges based on SED 17 of IABSE can also be adopted. In this approach bending moment, shear and axial forces of the deck in ULS check could be calculated assuming that all the cables have reached the conventional yielding point 80% GUTS and permanent loads and live loads are factored as usual (fig. 7.3)

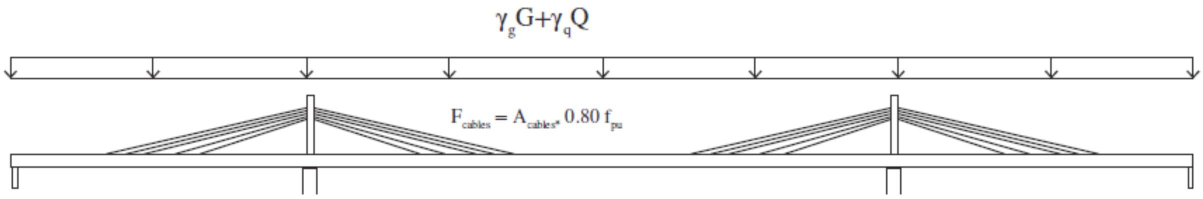


Fig 7.3: Simplified analysis at ULS for extradosed bridge

Section III: Design of Extradosed Cables

7.22 *Design of Cables*

The design of cables for extradosed bridges must comply with the requirements of this section.

7.22.1 *Cable Properties*

Guaranteed ultimate tensile strength.

For parallel-Strand or Parallel-wire stay cables the Guaranteed ultimate tensile strength of the cable F_{GUTS} is given by

$F_{GUTS} = f_{GUTS}$ x n x A_p

Where,

n is the number of strand/wires

A_p Area of each strand/wire

f_{GUTS} Guaranteed ultimate tensile stress of the cable

7.22.2 *Ultimate Limit State for Cables*

The tension of the cable under ULS load combinations during the construction phase and operation must satisfy following

$$\mathrm{F}_{ed} \leq \frac{F_{GUTS}}{\gamma_m}$$

γ_m - 1.25,

All Anchorage parts (Anchor Blocks, Socket case, Jaws, Swaged Sleeves etc) and the adjustment parts between cable anchorage and the structure should be designed to be stronger than ultimate strength F_{GUTS} of the cable these are to anchor.

In addition, under seismic or Accidental loads, there must be no irreversible plastic deformations on the parts which might hinder cable stay removal.

The parts of the structure on which the cable stay bear, and which transfer cable tension to the structure should be designed for Limit State condition as per the IRC and also must be able to take force of 1.0 F_{GUTS} under ULS condition.

7.22.3 *Seismic Design for extradosed cables*

All cables need to be designed for earthquakes, particularly to avoid plastic deformations in the cable (values to be limited as per section 7.21.2) taking into account flexural effects under large deformations of the structure.

Saddles that depend solely on friction may not be appropriate for use in seismic zone IV and zone V, without detailed structural investigation of the bridge, such as time history analysis, using relevant ground motion input for the site and friction properties of the saddle.

7.22.4 *Serviceability Limit State for Cables*

i) ***During Service***

The extradosed cables should be checked for maximum stresses in rare combination which is related to the stress range of the frequent live load in the cable as per following equations. The frequent loading does not include S.V. loading but includes congestion factor as per IRC 6.

$f_{sls} <= 0.6\ f_{GUTS}$

$D_{sls} < 50$ Mpa

f_{sls} - maximum permissible stresses under rare SLS combinations

D_{sls} - stress range in cables due to vehicular live load under frequent loads (MPa)

f_{GUTS} - Guaranteed Ultimate tensile strength of the stay cable.

For Extradosed bridges, if all the cables have D_{sls} (as explained above) less than 50Mpa, then, the qualification requirement of the extradosed stay cable can be adopted and additional fatigue check is not required.

If D_{sls} in any cable is more than 50Mpa then all the requirements of the stay cables including mechanical qualifications (including cable testing), SLS design, ULS design and Fatigue Check should be adopted. For design requirements of stay cables, Guidelines for the cable stayed bridges should be followed.

ii) ***SLS check under Construction and cable replacement.***

The permissible axial stresses in cables during construction and cable replacement under rare SLS load combinations should be as below.

$f_{sls} = 0.6\ f_{GUTS}$

References

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SECTION 8

EXTRADOSED CABLE SYSTEM, MATERIALS AND QUALITY ASSURANCE

In the initial stages of the evolution of extradosed bridges, in majority of cases external cable technology was used for extradosed cables. There are also instances of mixing of stay cable technology with the external prestressing technology to provide a durable system for the design life of the bridges in addition to low cost and savings.

The extradosed tendons are those tendons that are situated somewhere between external tendons and stay cables. Important characteristics for the performance of the tendons are the maximum stress in the tendon, the stress range under fatigue loads, the angular rotations at the anchorages and corrosion protection.

Table.8.1 : Summary of cable characteristics

Type of cable	Maximum stress	Fatigue stress range in Mpa	Angular rotation at anchorage (mrad)
Stay	40-50% GUTS	About 120 MPa (between 80 MPa-160 MPa)	10
Extradosed	50-60% GUTS	About 50 MPa (between 30 MPa-80 MPa)	2-5
External	70-80% GUTS	About 15 MPa	0

8.1 General Requirements

8.1.1 *Principles for Extradosed Cable Protection*

The basic principle for an extradosed cable protection means that the extradosed cables have to be protected against fretting and corrosion of the tensile elements so that long service life is guaranteed before replacement becomes necessary.

Design life of extradosed cables is always lesser than service life of structure. Considering the difficulties in replacement of extradosed cables, it is necessary that extradosed cables require to be replaced only once in the entire service life of structure with appropriate maintenance during this period.

8.1.1.1 *Fretting –resistance Precautions*

The main tensile elements of which extradosed cables are made may be in contact with each other and with other steel parts, which may occur in the free length (contact between individual wires) and in anchorage zones or zones of attachment of the stay or saddle zones. As a result of microscopic and frequent movements, ultimately loss of material occurs at these contacts. wear (called fretting corrosion) and fatigue cracking (called fretting fatigue) which must be prevented by providing an interface material which either mechanically isolates or lubricates the contact between the steel parts in all zones of cables.

8.1.1.2 *Redundancy of corrosion –protection barriers*

The corrosion protection of the steel tensile elements of extradosed cable must be provided with a degree of redundancy by means of two complementary (Fig 8.1), nested barriers:

- The first barrier (or internal barrier) is a protective coating applied directly to the main tensile element, and must cover the full length of the cable in all zones without a break.
- The second barrier (or external barrier) consists of an outer casing and an intermediate medium between the outer casing and the internal barrier. To prevent the internal barrier being consumed, the outer casing must be completely airtight and watertight, in both the free length and the anchorage zone. In addition, the intermediate medium must prevent any water or moisture that might get inside the external barrier from reaching the internal barrier.

Any damage caused to second barrier should not expose the main tensile elements directly to environmental aggression, thereby justifying the requirement of the internal barrier.

8.1.1.3 *Continuity of corrosion-protection barriers*

The two corrosion-protection barriers as described above must be continuous throughout the length of the stay, and especially in the anchorage zone.

8.1.2 *Internal barrier*

Sacrificial metal coating layer such as a galvanized coating is considered as the best internal barrier.

In most cases, the internal barrier must meet the dual requirements of corrosion protection and protection against fretting. Experience with galvanization demonstrates the favourable role of zinc not only as sacrificial protection against chemical corrosion but also as a contact interface whose wear-induced particles do not present the same drawbacks as rust from steel-on steel contact.

8.1.2.1 *Protection by metal coating*

The quality and lifetime of this internal protection depend on the type and thickness of the metal coating.

The following criteria are to be considered when choosing the thickness of the metal coating:

- provide sufficient thickness to prolong the lifetime of the sacrificial protection

- comply with the technological limits of the interface between the steel and the protective metal coating (adherence, continuity, ductility, etc.)
- for stays consisting of strands anchored with split-cone wedges, avoid excessive coating thickness as too much zinc could clog the teeth of the jaws when the strand is pulled up using stressing equipment
- for multi-layer strands with round wires, ensure that the metal coating does not compromise the strength of the wires

The protective metal coating must be applied to individual wires as they are manufactured.

8.1.2.2 *Non-metallic protection equivalent to galvanization*

Instead of a galvanized metal coating, a protective coating using new materials (such as epoxy resin) may be used, subject to it being demonstrated that the coating has a level of performance equivalent to that of galvanization in respect of the following properties:

- continuity and adherence to the support under static (bending) and dynamic action effects in service;
- lubrication of contact points to prevent fretting corrosion and fatigue;
- corrosion protection.

8.1.3 *External barrier*

The external barrier can take a variety of forms, in accordance with the type of extradosed cable and special requirements specific to the structure. It is generally designed and erected by the proprietor of the stay system employed, who must define the following in his technical literatures.

- The type of external barrier for the free length of the extradosed cable (outer casing and intermediary medium);
- The arrangements for ensuring the continuity of the barrier throughout the anchorage zone.

8.1.3.1 *Watertight outer casing*

For the free length of the extradosed cable, the outer casing is usually a sheath of one of the following kinds:

- stay pipe made from plastics tube: polyolefins, high-density polyethylene (HDPE), or similar
- plastics sheath extruded directly onto individual strands, in the case of individually sheathed multi-strand extradosed cables
- steel stay pipe made from pipe sections welded together; these are either protected against corrosion or are made of stainless steel.

Casing, paint or epoxy coating of multi-layer strand extradosed cables can give adequate protection, however it does require demanding periodic maintenance.

When the external corrosion-protection barrier is a stay pipe, it can also fulfil other functions, particularly:

- Mechanical functions (limiting cable-stay drag or rattling of main tensile elements);
- Aerodynamic functions (limiting rain & wind induced instability);
- Aesthetic functions (colour).

8.1.3.2 *Intermediate medium*

In the free length of the extradosed cable, the intermediate medium is generally:

- either a blocking medium in the annular space, using a suitable infilling product (generally wax, grease, or resin);
- or a permanent controlled-humidity air flow around the main tensile elements, using a cable dehumidification protection system. Such a system prevents condensation between the two barriers.

8.2 Extradosed Cable Types For Stay Cable Technology

In earlier days of construction of cable supported bridges, parallel strand system with Outer HDPE pipe injected with cement grout (as an intermediate medium) were used. However, the system has been prone to several problems:

- Cracks in the Cement grout due to shrinkage and bending of stay cables.
- Difference between the coefficient of expansion of the cement grout and HDPE was source of disorder.
- Between interface of cement grout and main tensile steel, fretting corrosion was observed. In fact, fatigue tests performed have revealed traces of iron oxide at main tensile steel -grout interface.

Extradosed cables are made up of different kinds of tensile steel elements and can be broadly grouped in following two categories based on strand system or wire system.

- Parallel Strand System-PSS
- New Parallel Wire System-NPWS

In majority of all recently constructed extradosed bridges, Parallel strand cable (PSS) has been used.

8.2.1 *Parallel Strand System (PSS)*

The main tensile elements of PSS Cables are group of strands, wherein each strand is made up of seven ply wires. Each wire is of 5mm in diameter except the central wire which is of larger diameter. Tensile strengths of such strands available in market are, 1860 MPa for 15.2mm and 1770 MPa / 1860 MPa for 15.7mm dia. [6]. However, 5mm wires with higher tensile strength of 2000 MPa has been also developed.

All wires of the strand are galvanized and such galvanised strands are extruded in individually sheathed HDPE tube & injected with wax or grease. All such individually sheathed strands are enclosed in an outer sheath or pipe.

Generally, diameter of outer HDPE duct is governed by numbers of strand grouped in it. Required diameter of the duct for PSS cable is more than other PWS cable. Larger diameter of outer HDPE results in more wind loading on the structure, which is an important issue for its design. In the last decade, stay cable suppliers have developed new PSS cable system of threading of strands, wherein strands can be accommodated in more compact manner and thus reducing the void-ratio inside outer duct.

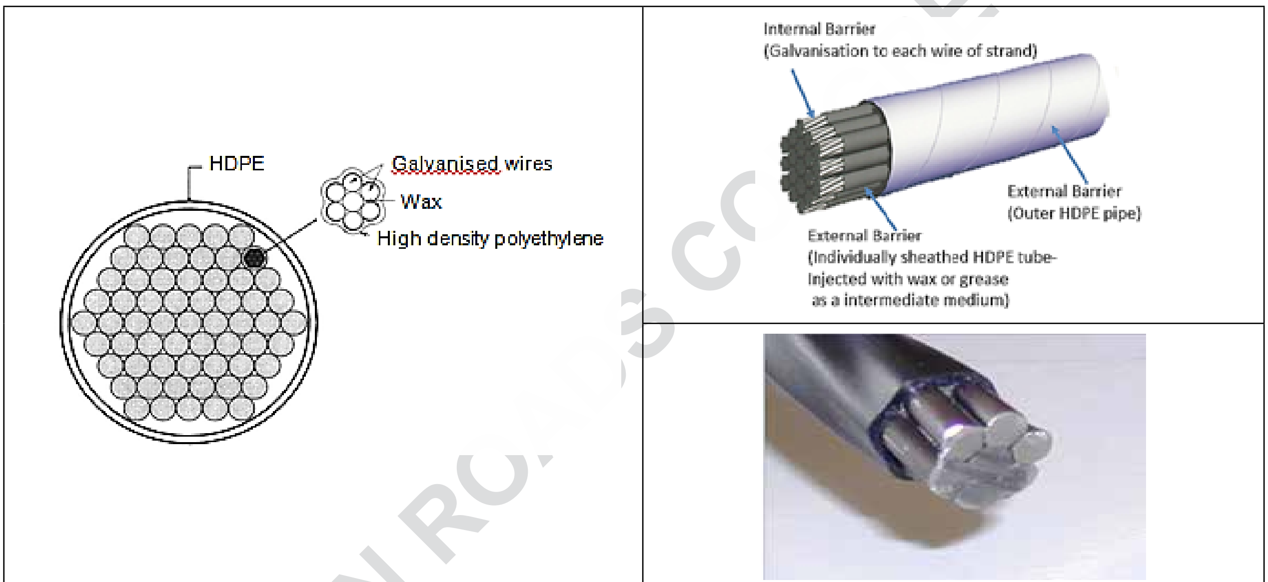


Fig-8.1 Cross section of Parallel Strand System (PSS) cable

A cable with 156nos of 15.7mm strand having a maximum breaking load of 41.3MN has been used in Maumee River Crossing in USA.

8.2.2 *New Parallel Wire System (NPWS)*

NPWS cables are a development of PWS cables, which was developed and used in Japan in the 1970s and 80s. These are composed of a wire bundle. For the NPWS cables, wires are slightly twisted with a long lay to ease the reeling and unreeling and make the cable self-compacting when subjected to axial tension. Besides the slight twist the NPWS cables are also characterised by having the protecting outer polyethylene cover extruded directly onto the wire bundle so that no void will exist between the wires and the surrounding pipe.

The wires used for NPWS cables are of 7mm diameter with tensile strengths of 1570 MPa – 1770 MPa.

The wires are protected by galvanisation and the wire bundle is wrapped in filament tape as well as the extruded high density polyethylene (HDPE) outer casing.

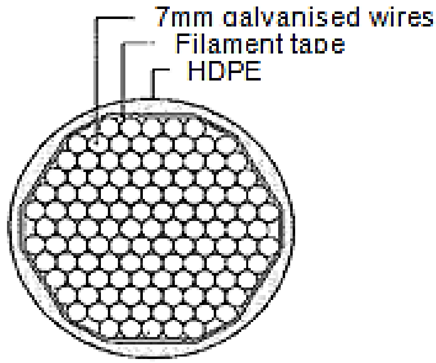


Fig-8.2 Cross section of New Parallel Wire System (NPWS) cable

NPWS cables are fabricated in sizes ranging from 19nos (7mm wires) to 499nos (7mm wires).

Diameter of whole cable ranges from 45mm to 192mm and the maximum breaking load for a 192mm NPWS cable is 34MN based on 1770MPa tensile strength.

8.3. Qualification Criteria and Testing of Extradosed Cables System

Three different levels of testing of extradosed cables are recommended [2]:

- Initial approval (qualification) testing of the extradosed cable system
- Suitability testing of the extradosed cable system for a particular project
- Quality control testing of the extradosed cable components for a particular project

The extradosed cable has three parts, viz the lower end anchorage, the upper end anchorage or possibly saddle, and the MTE (Main tensile elements). All these elements are influenced by the variation of stresses in the strands. As the length is short, the extradosed cables are less sensitive than in a cable stayed bridge, to fatigue effects and also to dynamic vibrations which also induce fatigue effects. Moreover, in the majority of Extradosed Bridges, the stress variations are less significant than in a cable stayed bridges.

In view of this, despite of adopting stay cable technology for extradosed cables, the qualification and testing criteria for extradosed cable system is less stringent compared to that for cable stay bridges. Upper Stress and fluctuating stress range for testing of cables are different for stay cable bridges and extradosed bridges [2].

If, for a particular project all the cables have $D\sigma_{sls}$ (stress range in cables due to vehicular live load under frequent live load in MPa) is in between 15 to 50MPa, then, the qualification testing (initial and suitability testing) requirements of the extradosed bridges shall be adopted. If the same is more than 50MPa, all the requirements of the cable stayed bridges shall be adopted (refer Annexure A5).

8.3.1 Laboratory and Independent Witnesses

Initial approval testing or suitability testing shall be executed in an independent laboratory which fulfil the requirements provided in EN ISO/ IEC 17025 or equivalent standard. In addition following requirements shall be considered :-

- The laboratory shall be independent and have enough international experience with testing of cable systems.
- The testing machines and measuring equipment's shall be calibrated according to ISO 7500 or equivalent standards. The calibration certificates shall be included in the test report. The uncertainty of the values measured with the measuring equipment shall be within + / - 1.0%.
- The laboratory shall provide automatic system to detect failures of tensile elements during fatigue testing.
- The laboratory shall observe all safety measures for safety of their workers as well as the machines.

Suitability testing can be also performed in private facilities which fulfil the same requirement as independent laboratories provided that critical testing steps such as installation of the test specimen , implementation of testing parameters, tensile tests or friction tests (wherever applicable for saddle) and reporting are witnessed by an independent body or expert , both to be approved by the owner.

All test procedures, methodology and acceptance criteria of test shall be submitted by supplier to owner prior to the test and the same shall be approved by owner.

8.3.2 Initial Approval Testing (System Qualification testing)

The objective of initial approval testing is to demonstrate the feasibility (practicability) and performance (reliability) of a proposed extradosed cable system design comprising a range of extradosed cable sizes and using the proposed materials.

Extradosed cable system supplier tests conducted in last 5 years are considered valid as long as there are no changes in relevant extradosed cable details.

For initial testing approval of anchorage / saddle axial fatigue and tensile test, stress variation ($D\sigma_{test}$) range of 200MPa with upper range of 0.45 f_{GUTS}, with two million load cycles shall be adopted, if all the cables in the project have $D\sigma_{sls}$ more than 50MPa. Otherwise, stress variation ($D\sigma_{test}$) range of 140MPa with upper stress of 0.55 $f_{GUTS,}$ with two million load cycles shall be adopted, if all the cables of project have $D\sigma_{sls}$ less than or equal to 50MPa.

Initial approval testing of extradosed cables anchorage assembly shall include :-

- Three Anchorage to be tested for axial fatigue and tensile tests performed on typical sizes (one small, one medium, one large) [2]
- One leak tightness test on a small cable size (after the fatigue test) [2]

Similar initial approval testing for saddle system (if adopted in project) with isolated extradosed cable shall include :-

- One complete fully assembled extradosed cable specimen with saddle setup for saddle axial fatigue, friction and tensile test [2]
- Effective Saddle friction Coefficient test [2]
- One complete saddle leak tightness test [2]

Once a particular extradosed cable system supplier has performed and qualified all acceptable limits of tests as per relevant code of testing for extradosed cable / saddle (if adopted in project) has presented test results for approval for incorporation of the system in the project, it shall be deemed to have been approved subject to the condition that all required tests during suitability testing shall also be satisfied.

But, any change in material specification, structural details or leak tightness details or corrosion protection barriers details will require re-testing of fatigue and tensile tests or leak tightness tests by supplier for initial approval.

If the supplier of a particular component is changed with the same material specification and structural details, only suitability testing is recommended to be done.

8.3.3 Suitability Testing

Basic materials, components and detailing of the extradosed cable system such as strands, pipe and anchorage components shall satisfy the same specifications as the components used for the approval testing. However, they may be fabricated by different suppliers or process than those used during initial approval testing.

The objective of suitability testing is to demonstrate the satisfactory performance of these materials and components of a extradosed cable system fabricated by a specific supplier and /or a process and intended for use on a particular project.

For suitability testing approval of axial fatigue and tensile test, stress variation ($D\sigma_{test}$) range of 250MPa with upper range of 0.45 $f_{GUTS,}$ with two million load cycles shall be adopted, if all the cables of project have $D\sigma_{sls}$ more than 50MPa. Otherwise, stress variation ($D\sigma_{test}$) range of 180MPa with upper stress of 0.55 $f_{GUTS,}$ with two million load cycles shall be adopted, if all the cables of project have $D\sigma_{sls}$ less than or equal to 50MPa.

Suitability testing of a extradosed cable system is recommended to be done for each project (based on size of cable being used in project) which include :-

- A minimum of three tensile tests on single tensile elements (strand alone, sample chosen from the source supplier of strand for the project) per batch using laboratory wedges to asses actual strength of the strands (AUTS) [2].
- One extradosed cable-anchorage assembly, with wedges and cables using the same system as intended to be used in the project for axial

fatigue and tensile test for the size of the cable to be used in the project. If the variation in the sizes of cables is large in a given project then, three tests may be performed instead of one, one with smaller size, second with medium size and another with larger size) [2].

Similar suitability testing of cable system using saddle (if adopted in project) is required to be performed which include :-

- Extradosed cable-Saddle fatigue and tensile tests using saddle similar to as carried out for Extradosed cable-anchorage axial fatigue and tensile test [2].
- One friction test per project [2].

For each test, 3 samples shall be selected.

If the suitability test does not satisfy the acceptance criteria for a particular material batch, the cable supplier has the choice between the following two options :-

- Replace such batch of material and repeat suitability test associated to the new material batch.
- Do not replace such batch of material and perform two more tests. If either of the two additional tests fails, the batch of material represented by the three samples shall be rejected.

This testing is time-consuming and should be initiated at the early stages of the project to avoid delays. All the testing procedure must be first submitted and approved by the Engineer. After the testing procedure is approved, testing may start, subject to the availability of the special laboratory equipment.

Above tests shall be carried in a sequence as mentioned above. Failure of any of the above test, may require re-testing of all previous tests.

Extradosed cable anchorage components have to be installed, while casting the segments. It is, therefore, preferable to have the testing completed prior to casting extradosed cable segments.

Apart from above, other material checks on strands, PE sheathed duct, filler material and HDPE duct, wedges and anchorages parts shall be also carried out [2].

8.3.4 Quality Control Testing

The objective of quality control testing is to determine that the properties of materials and components installed in the extradosed structure are equivalent to the properties of the materials which were used for initial approval and suitability testing.

Based on agreement between supplier and client for lot supply and test frequency , all / some of tests as specified in Annexure-A5 for testing of various material checks on strands , sheathed duct, filler material and Outer HDPE duct, wedges and anchorages parts are required to be performed [2].

8.4 Cable Vibrations and Control

As extradosed cables tend to be short and as these are working at higher tension levels than stay cables, they are less prone to dynamic vibrations. Their periods of vibration are shorter than in stay cables. Often, the vibration effects are neglected.

The risk of vibration increases with very high extradosed cable length. Short extradosed cables (less than about 70m-80m) generally involve no risk. For long spans of extradosed superstructures such as 250m, wherein possibility of parametric resonance may occur, internal dampers may be required to reduce the amplitude of cable vibrations.

The most conventional way of limiting or eliminating vibration involves increasing structural damping capacity by fitting special damping devices. The objective of a damper is to reduce the amplitude of vibration. However, the free longitudinal movement and free rotation of the extradosed cable at damper location is allowed for temperature elongation and force variations of the cables without any constraints.

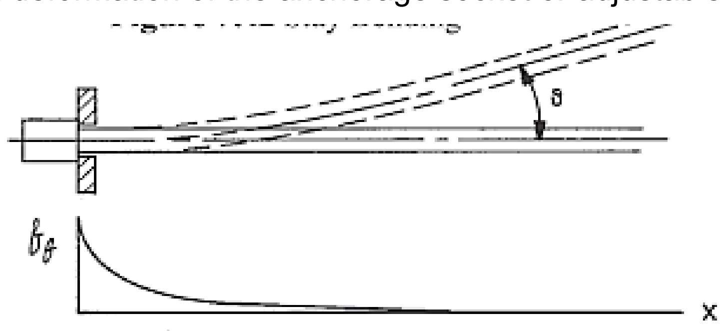
Many types of dampers for extradosed cables as required for larger spans are available nowadays, for example, rubber dampers, viscous dampers, friction dampers, oil dampers and so on. In order to derive suitable performance of dampers, optimization of extradosed cable dampers is needed depending on the frequencies and amplitudes of vibration to be mitigated. The designer must discuss with suppliers to define the best solutions in accordance with the risks.

8.5 Bearing Plate, Recess Pipe Installation

The extradosed cable recess pipe is installed when casting the segment. Normally, the recess pipe is bolted or welded to the anchoring bearing plate at a right angle.

Great accuracy is required when placing the guide pipe as it will dictate the extradosed cable alignment.

The vertical and horizontal angles are especially critical, as offsets would create bending stresses (Fig 8.3) in the extradosed cable. The anchorage adjustable nuts or shims are in contact with the bearing plate. The bearing area must be perfectly flat to avoid stress concentrations and deformation of the anchorage socket or adjustable nut.



$$\sigma = 2\alpha \frac{d}{4\lambda}\sqrt{\frac{EF}{A}} \times e^{-KX} \qquad K = \sqrt{\frac{F}{EI}}$$

Fig 8.3 : Stay Bending

8.6 Extradosed Cable Pipe Installation

The main difference in installing the various cable types occurs on the basis of whether the cables are erected as fully prefabricated units (NPWS) or are site-assembled units erected strand by strand (PSS).

Erection of fully prefabricated cables will require transport and lifting of heavier units and deployment of heavy lifting equipment.

Erection of PSS cables requires only light equipment, but is considered to be slightly more time consuming.

As the cable installation is not normally on the critical path, the main difference from a costing point of view between the systems is the plant associated with the installation process, where the lighter equipment used for the PSS cables is a clear advantage.

Steel pipes have to be butt welded. Many of the welds can be done with the pipe lying flat on the deck, while others have to be done with the pipe in an erect position. In either case, the detailed butt weld procedure must be developed and approved. Random x-ray check of the welds is recommended.

Polyethylene (HDPE) pipes are welded together by fusion, using a special device to maintain proper pipe alignments.

Different systems are used to temporarily support pipes prior to strand installation and extradosed cable stressing. They all have to be designed to prevent excessive bending of the pipes. The extradosed cable pipes can be supported from the previously erected pipes, with messenger cables or highline, or by a few extradosed cable strands inside the pipe.

8.7 Installation & Stressing of Extradosed Cable & its Components

Extradosed Cable supplier instructions should preferably be followed for the installation of the other system components too. Guidelines for the same are enumerated in section 6.3.7.1.

8.7.1 *Stressing Of Strands*

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

(a) Single stressing system using monojacks (Isotension method)

(b) Multistrand jack stressing (gradient jack).

8.7.1.1 *Single stressing system Using Monostrand jack (Isotension method)*

Parallel strands stay cables are installed normally strand by strand. Due to flexibility of deck and tower, the force of the individual strand of a stay cable varies during the stressing operation. To ensure uniformity of force in all strands of a stay at the end of stressing operation, a method has been developed by Freyssinet, called Isotension. The principal of the Isotension method may be described as follows:-

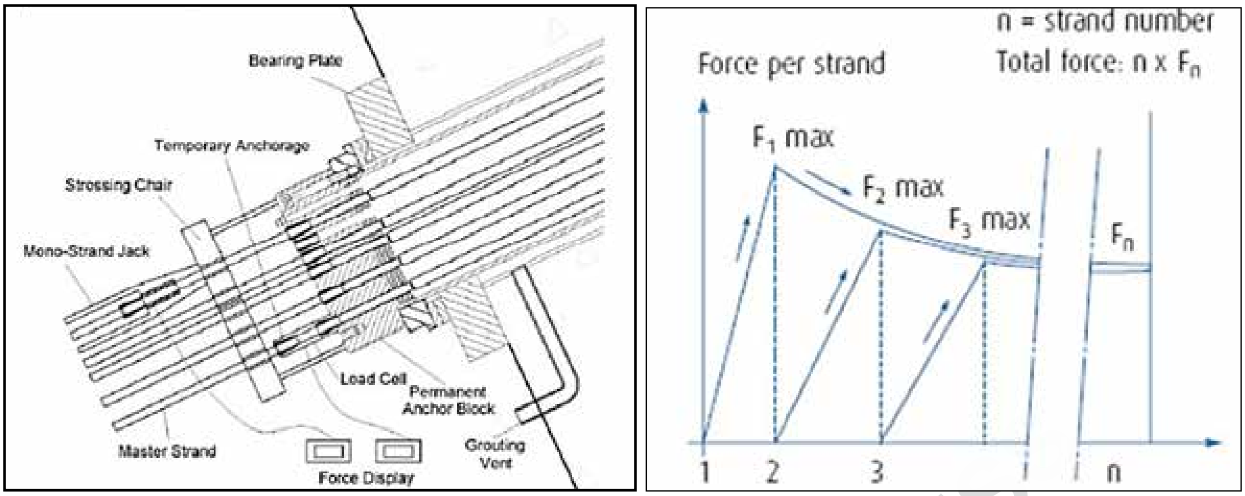


Fig 8.4 Isotension Method and principle diagram

- The first strand ("the master strand") is fixed to one of the stay cable anchorages and its other end is threaded into the other anchorages. It is then cut and stressed to a pre-calculated force which has to be higher than the desired target strand force (when all strands of the stay cables are stressed). Such pre-calculated force needs to be arrived at based on an analytical model taking account of flexibility of superstructure and tower.
- The wedge of the master strand is not yet engaged to the stay cable anchor block. The master wedge is instead wedged to a special single anchoring device provided with a load cell which gives the reading of the current master strand force.
- The second strand is then installed in similar manner, cut and stressed. The stressing is done as for the master strand, with a mono-strand jack equipped with a load cell identical to the one indicating the force of the master strand. As the second strand is stressed, the force in first strand decreases and the stressing operation is stopped when the readings of the two cells are identical. The second strand is then anchored/ wedged to the permanent anchor block. The two strands are equally stressed.
- The third strand is then installed and stressed until its force reaches the force of the master strand (which decreases jointly with the force of the second strand). The three are identically stressed.
- The same operation is repeated until the last strand of the stay cable. The last reading of the stay is then to be compared with the target theoretical force.

8.7.1.2 *Multistrand Jack stressing (Gradient Jack)*

Final stay cable force may then be adjusted by the multi-strand jack, if necessary. Elongation of extradosed cable which can happen using multi-strand jack is very limited in the stressing end anchorages design and its requirement must be defined by designer in the beginning of job to the anchorage supplier.

8.8 Cable Replacement

Replacement of the NPWS cables has to be done cable by cable and substantial traffic restrictions will be required for a considerable length of time. Handling of the cables will require the use of heavy lifting gear, both at deck and tower level.

Replacement of PSS cables can be done strand-by-strand using basically hand held equipment requiring only limited or no traffic restrictions.

PSS cables have a clear advantage when comparing replacement procedures and as it is usual to assume a 50 to 60 years design life for the cable system. At least one total replacement of the cable system during the life of the bridge must be assumed.

8.9 Protection from fire and Vandalism.

Vehicle fires can happen occasionally on bridges. Since bridges are in open area and well ventilated, hence they are relatively less exposed to high temperature rise in the event of fire of a truck/car. However, if the truck carries hydrocarbons and catches fire, it could cause a significant rise in temperature of a bridge in the event of fire. If the truck burns near a extradosed cable, structural stability of structure must be ensured by considering the failure of one extradosed cable.

However, the case may be different for an extradosed bridge with set of closely grouped cables or large number of trucks carrying hydrocarbon (near oil refineries) which cover a large number of extradosed cables. In such special condition, provisions needs to be made so that temperature rise in the main tensile elements of the cable / anchorages be essentially retarded for a period, it would take to extinguish the fire.

Study of vehicle based fire hazard may be required to be carried out to asses required fire rating of extradosed cable system. Such study includes:-

- Fire characteristics of vehicles intended to be passed over bridge
- Heat transfer to the structural elements via radiation heat flux
- Temperature increase of the structural elements
- Resulting material and mechanical response to the elements
- Range of fire type, size, location to develop envelope of performance
- Effectiveness of fire protection measures
- Fire Ventilation

Special insulating material have been developed by extradosed cable suppliers for such purpose which enhances fire resistance to main tensile element (MTE) by protecting MTE to a level so that it can carry applied design load. Fire resistance qualification tests shall be required to be carried out to establish fire rating of extradosed cable system [3]. The

30-minute fire rating for stay cable including anchorages is a minimum standard for fire resistance. However, if the owner feels fire rating demand for a project specific is more than 30-minute, the extradosed cable system supplier needs to demonstrate with a longer duration of fire rating.

It may be noted that in case of fire event, the entire cable system needs to be replaced irrespective of usage of such insulating material of required fire rating.

The design of transition pipes must allow for special steel pipes protecting the cable against vandalism also. The details shall be designed such that these steel pipes do not impose force or deformations to the cable which could be harmful for the extradosed cable performance.

8.10 Anchorages for Cables

The two ends of extradosed cable are called as anchorage zones which consist of:

- Anchorage which transmits the tension of the extradosed cables to the attachment parts of the structure (deck, cross beams gussets, pylons etc..)
- Transition zone extends from the anchorage to the start of the free length of the extradosed cable. The transition zone comprises of deviators, transverse guide systems, external dampers and sealing system.

  The anchorage zone and the transition zone generally fulfil the following functions:-
- Transfer of Extradosed Cable Force- Anchorage consists of mechanical part (wedges) designed to secure the main tensile elements of the cable and transmit their forces to the attaching parts of structure.
- Filtering out Angular Deviation- Anchorage may comprise cable guide systems in order to partially or totally prevent angular deviations of the cables extending to the anchorage head. Hence appropriate system be provided to limit or eliminate the effect of extradosed cable deviations at the anchorages head i.e to filter out changes in angle between the cable end and anchorages.
- Possibility of Adjustment- Anchorage has provisions for moving the anchorage head with respect to structure to provide extradosed cable tension adjustment and initial extradosed cable directional adjustment.
- Corrosion Protection and Water Tightness- The design of anchorage must allow the two barrier corrosion system of strands to be extended up to anchorage plate without any break. Certain layer of protection specified for free length can not be fully continuous from end to end of cable. This applies in the transition zones wherein sheathing of prestressing steel is removed inside the anchorage. Whenever one of the layers of protection

is interrupted it must be replaced with an equivalent layer of protection.

- Provisions shall be made in the anchorage to drain out water to clear out ingress through any stay barrier or stay-anchorage connection
- Cable Replacement- The design of anchorage allows the replacement of complete extradosed cable as a whole (for NPWS cables) or in parts strand by strand (for PSS cables).

8.11 Anchoring / Passage of Stay Cables in Pylon Body

Two dispositions exist for the transition through the mast head. The first cuts the continuity of the extradosed cable. In this case there are 3 typical cases which are defined in Fig 8.5 as cases A, B, C. The second keeps continuity of stay cable within the saddle, shown as case D.

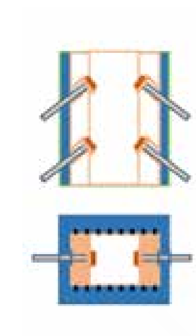
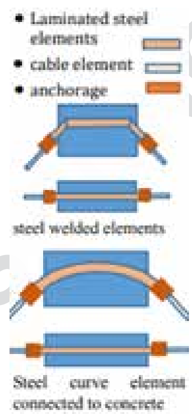
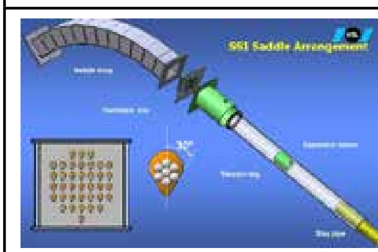
Type	A	B	C	D
Elevation Plan				 Courtesy-M/S Freyssinet International  Courtesy-M/s VSL
Comments	Cables anchored inside concrete pylon	Cables anchored inside composite element or in Mast head steel	Cables anchored in cast-in steel element	Individual holes for individual strands of cables passing over saddle in concrete pylon

Fig 8.5 Anchorage configuration in pylon

The A and B type of steel anchorage box structure at the head of pylon allows an easy maintenance as such are extensively adopted in cable stay bridges. During construction, methods that enable the erection of this anchorage system are limited. The working space which is necessary for installation of anchorages in pylon increases its size and also all the equipment for maintenance: access, stairs, platforms. The C type of anchorages of extradosed cables at pylon can be installed externally using laminated steel elements installed in the pylon head before pouring of concrete is completed. This is done through an external nose element in steel on which the cable is anchored. After stressing of stays, the tension in internal steel link saddle creates an inward pressure on the concrete body of pylon head.

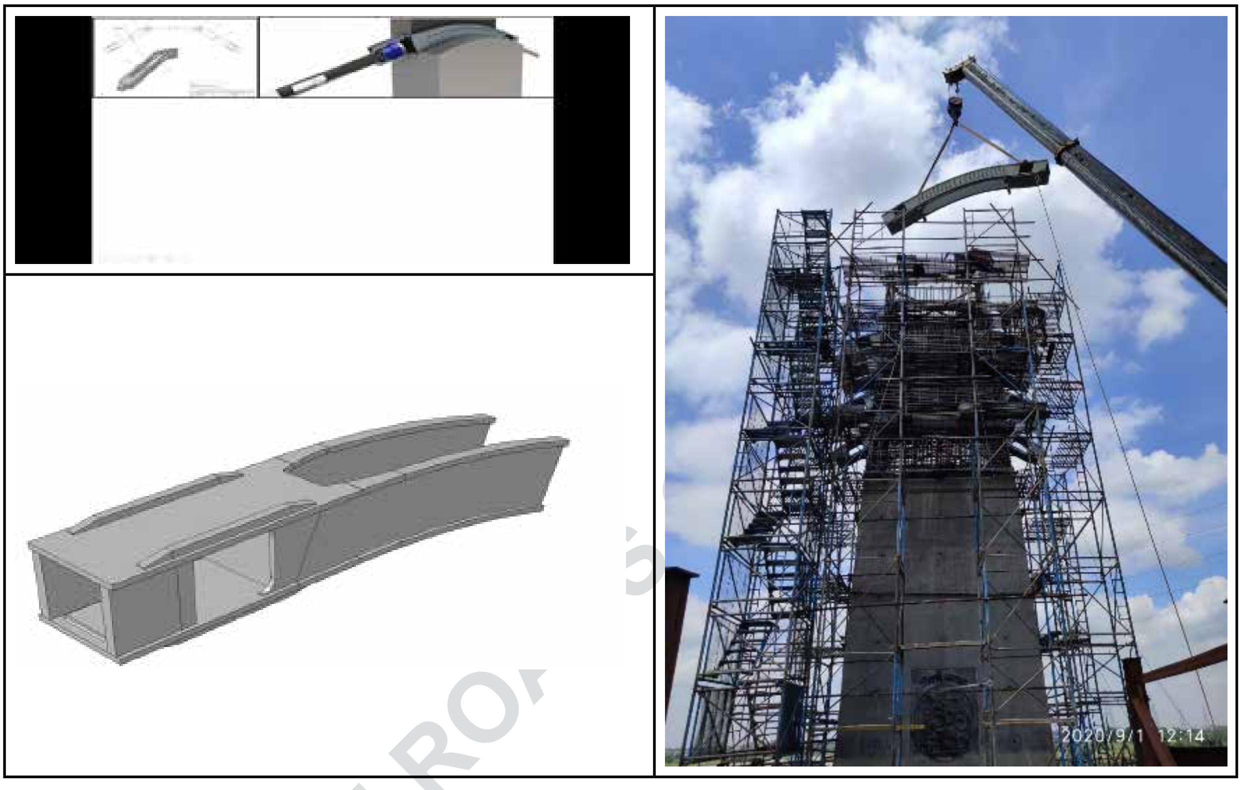


Fig 8.6 Internal steel link type saddle

The Extradosed cable is not continuous and is anchored on each side, on an external nose section where the cable can be anchored by means of standard anchorage. Lateral openings can be fitted to allow an easy replacement of single strand or even whole cable if required (Fig 8.6).

In concrete pylon, such saddle have added structural advantages as: -

- Shear studs at the steel beams inside the pylon transfer differential loads easily within the concrete although a proper check of elongation vs shear stud loadings needs to be performed.
- Friction, bond stresses and mechanical wear due to strand deviation are avoided.
- An unbalanced installation sequence as well as the use of two different cable sizes at one anchor box becomes possible.

- The left and right side cables of one box can be differently inclined, and there are no limits in the respective deviation radii.
- The design of the anchor box is fully in accordance with conventional steel construction standards with element always in tension.
- If any inspection and maintenance of the corrosion protection at the cable anchorage is required, it can be performed easily from outside the pylon structure.
- Possibility exists to design a slender and elegant pylon
- There is no necessity of an access inside the pylon

In some cases, instead of anchoring the cable in pylon, extradosed stays are allowed to continuously pass thorough pylon by providing a saddle. These D type (Fig 8.5) saddles have been used in several extradosed bridges and are a preferred option for extradosed cables in non-seismic zones. These allow the continuity of tensile elements without break and limit the sizes of the pylons. These provide the anchorage for the cable while avoiding their slippage. There is one saddle for each cable. The advantages provided by saddles as reported generally are:

- Saving of anchorages
- No tension elements buried in concrete
- No necessity of an access inside the pylon
- A possibility to design a slender and elegant pylon

The design of cable saddles is an integral element of pylon conceptual design and the geometric layout of the tower head. The deviation saddle, usually made of steel, guides the cable deviation with the adequate curvature and supports the downward force resulting from this deviation. It provides a continuous cable stay from the bridge deck, through the saddle on the pylon and back down to the bridge deck on other side.

Saddles, which are solely depend on friction to block differential forces in stay cables on either side of saddle shall not be used in high seismic zones. If adopted, detailed structural investigation of the bridge shall be carried out using site specific response spectrum or time history analysis using spectrum compatible time histories.

Saddle is only an alternative option to replace the stay cable anchorages in pylon of relatively small and slender structures mainly for the reasons of space requirements. However for larger structures, wherein space requirements in pylon are not critical, anchorages at the pylons are always preferred for their proven performance in terms of tensile and fatigue strength. Anchorage also offers advantage in terms of ease of installation and possible future replacement which is very difficult in saddle system.

If at all, saddles are proposed by designers, it shall be ensured that:-

- Blocking of differential forces in extradosed stay on either side of saddle (pylon) by internal friction and transferred to pylon using mechanical anchorages. Slippage shall be avoided at any state of the structure including at ULS condition.
- Detailing of the pylon structure around the saddle using reinforcement, if needed according to the applicable design code to ensure the transfer and diffusion of the differential forces in the pylon body structure.
- Saddle system performance shall be at par with cable anchorages. The bending of the cable at the entry into the saddle under permanent and variable loads shall be adequately considered by providing guiding devices / bending filters.
- Continuity of corrosion protection of cables through the saddle shall be ensured or be provided by different protection system inside the saddle which is equivalent to the one provided in the free length. The transition of main to equivalent layer of protection has to be carefully detailed to ensure leak tightness.
- Replaceability of the extradosed cable
- Prevention of fretting corrosion in between main tensile element and saddle components
- Capacity to withstand fatigue loading due to pressure and friction of main tensile element against saddle components

Because of the curved profile of cables in saddle zone, flexural stresses in stays are often very high and need to be combined with axial tensile stresses.

For Saddle system, requirement of initial testing and suitability testing as given in section 8.3.2 & 8.3.3 respectively shall be followed.

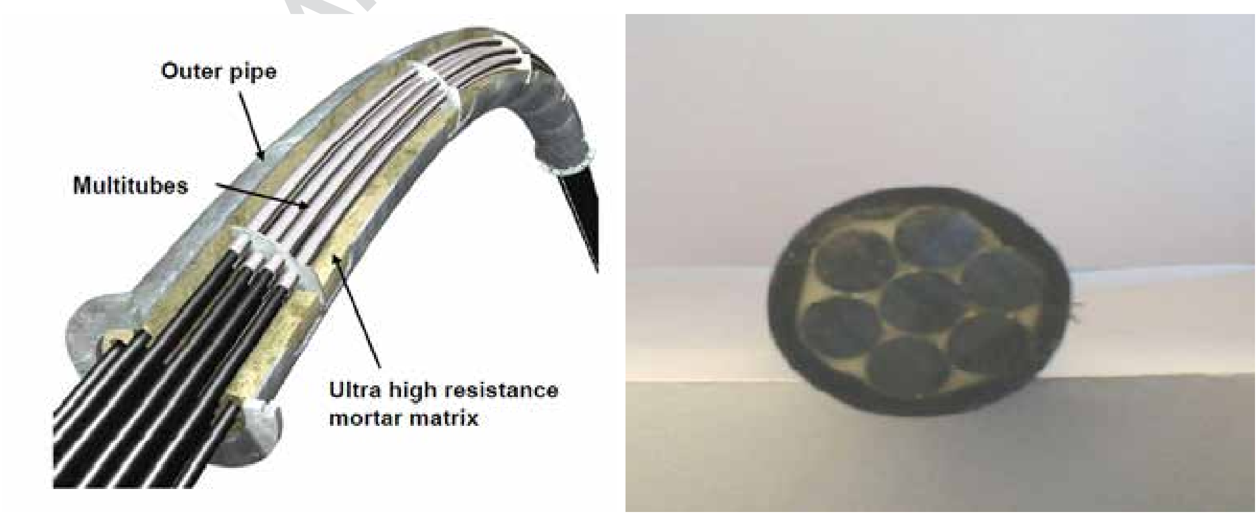


Fig-8.7 Multi-Monotube Saddle-Courtesy- M/s Freyssinet International

Conventionally, multi-mono tubes (Fig 8.7) saddle are used with closely spaced diaphragm and space around tubes is filled with high strength mortar. Each strand of cable is allowed to pass through each curved profile tube. However, in such cases, strands to be used are patented (sheathed strand with specially designed resin or epoxy coated strands) of system supplier's so that fretting corrosion of strand does not happen inside the tube.

Structural Saddles (Fig 8.6) in which main tensile elements are coupled at its both end shall be designed in accordance with standard applicable [5] and tested in accordance with conventional stay-anchorage saddle system requirements.

8.12 Levels of Protection For Preventing Corrosion Of Extradosed Cables.

8.12.1 *Corrosion Protection Of Extradosed cables*

The corrosion protection for the tensile steel element (the wires) must be provided with a certain redundancy in the form of two complementary nested barriers.

The first barrier generally employed in all systems is a hot dip galvanic coating applied directly to the wires and covering the full length of the cable without a break.

8.12.1.1 *Parallel Strand System (PSS) Cable*

For PSS cables each 7-wire strand is individually sheathed in an extruded HDPE. A corrosion-inhibiting compound, usually in the form of wax or grease, fills up the voids between the galvanised wires inside the sheath. The galvanisation and the extruded HDPE make up the two complementary barriers. In addition, the strand bundle is usually enclosed in an outer HDPE tube to give a smoother surface for the cable. Although this outer pipe constitutes an extra layer of protection it cannot be considered a complementary barrier as water can travel freely along the cable in case the tube is damaged.

Prior to the introduction of the individually sheathed strands a commonly used method was injection into the void between the galvanised strands and the outer HDPE tube with a corrosion-inhibiting compound, usually in the form of cement grout. However, a number of extradosed cable failures with this type of protection have been observed over the years and this is no longer a recommended method.

8.12.1.2 *New Parallel Wire System (NPWS) Cable*

For NPWS cables the tightly packed galvanised wires are surrounded by an extruded HDPE sheathing. The wire bundle is wrapped in filament tape prior to sheathing of the HDPE.

In order to achieve two complementary layers of protection the NPWS cables relies on the filament tape to prevent water to penetrate along the length of the cable in case of damage to the outer HDPE layer. It is therefore debatable whether the NPWS system fulfils the two level of Corrosion requirement unless an active corrosion-inhibitor fills out the voids between the wires.

8.13 Protection of Extradosed Cable From Ice Accretion with De-Icing & Anti-Icing Systems

Cable supported bridges can suffer from ice and snow accretions that develop in specific weather conditions, predominately in cold climatic regions such as northern part of the country which are 2000m above sea level like Ladakh, Lahul-Spiti, Shimla, Manali. Ice generally cover the entire area including the cable/bridge structure.

High levels of humidity combined with frequent changes between passing cold and warm fronts results in high risk of atmospheric icing. These are formed from cloud droplets, rain drops, snow or water vapour. Duration of icing event is also important in development of ice accretion.

Pieces of accreted ice or snow fall from bridge members occurs

- when their weight increases beyond a value
- when temperature of outer surface of structure rises above 0oC , melting occurs at the surface-ice interface.

Block of Ice or Snow falling from extradosed cable / pylons members may result in traffic accidents which can cause direct damages to passing vehicles and human safety at risk. Consequently, lack of successful de-icing or anti- icing measures may result in bridge closure which leads to traffic hindrance that can in turn lead to severe financial losses.

Also, icing on bridges can result in fear / frightening to drivers, striking the vehicles while passing over a bridge as safety is not ensured until removal of the accreted ice or snow is complete. In some cases, traffic is suspended till operation is completed.

Denmark's Great Belt Bridge was closed for 12 hours per year during falling of ice. Oresund Bridge, Severn Bridges of UK and Uddevalla Bridge of Spain have suffered similar experiences.

As a safeguard, different de-icing and anti-icing techniques [4] developed which are applied to bridge cables or pylons and it can be broadly classified in to three categories:

- Mechanical methods based on breaking down of ice
- Thermal methods based on melting
- Passive methods based on natural forces

8.13.1 Mechanical Method [4]

In this method, the ice is removed manually or automatically usually by closure of traffic.

Manual or mechanical method is carried out by physically breaking off the ice which requires the workers to be lifted in gondola operated by crane along the cables. Such method can be only adopted, when there is no snow fall or string winds. Another way of manual method is by hitting hangers with base ball bat. This is being adopted in George Washington Bridge of New York.

There are many Automatic mechanical methods available. Electromagnetic-Impulsive De-Icing (EIDI) systems employs by inducing strong and sudden magnetic force from a high current DC pulse through a coil, resulting in rapid acceleration and flexure to icing surface. This leads to debonding and breaking the ice. When such techniques are employed by installing the coils along the exterior surface of the cables, physical outer diameter of cable is increased. This technique was used in Great Belt East Bridge.

8.13.2 *Thermal Method [4]*

All thermal method works on basic principle of heating the ice-prone surface to prevent formation of ice or to generate de-icing. All thermal methods consume large amount of energy when operated.

Indirect heating by applying warm air to heat the ice prone area so as to melt the ice or prevent it from forming. Such method was applied in Uddevalla bridge, Sweden where a high pressure system pushed warm air through a small opening in cable protecting pipe and therefore kept the whole cable strand at temperature above freezing. Such method is successful but demands very high energy.

Others thermal solution is Pulse Electro-Thermal De-icing (PETD) method which uses short pulses of electricity applied directly to the ice-material interface by covering the surface with a thin electrically conductive film which is heated with milliseconds long pulse creating a thin water layer causing ice to drop off. This was used in one part of Uddevalla Bridge where both a cable stay and part of pylon was covered in PETD and very cost-effective as micrometer thin layer of ice is required to be melted.

8.13.3 *Passive Method [4]*

Among many passive methods of anti-icing, none of the method completely hinders the formation of ice. However, such method can reduce the probability of formation of ice. Some of known methods are:

- Thermal absorbent coating which is effective only when sufficient radiation is available.
- Hydrophobic coating with surface tension and adhesion which is only effective in wet snow conditions.
- Solid icephobic coatings but still have ice-adhesion forces ten times greater than what is necessary for ice to detach under gravity or wind loading.
- Viscous products and anti-icing greases but have limited duration of protection as they loses their efficiency under the effect of precipitation.

8.13.4 *Conclusion*

To ensure traffic safety and to reduce long bridge closure, it is important to adopt a novel method against ice accretion and to mitigate the risk of falling ice and snow.

Passive system is not fully effective to mitigate accretion ice or snow.

Electromagnetic-Impulsive De-Icing (EIDI) system appear the most promising among mechanical techniques and worked very well when installed on Great Belt Bridge Hangers.

Although various thermal techniques are available but they consumes lot of energy. However, Pulse Electo-Thermal De-icing (PETD) system shows good energy efficiency.

It is high time to adopt EIDI or PETD system on cable supported bridges in snow clad areas.

8.14 Durability Design Of Extradosed Cable System

The durability of Extradosed cable structure can not be easily defined as each structure is unique of its own and factors governing the durability of extradosed bridge or extradosed cable system are:-

- General design of structure
- Service condition
- Dimension of structure and particularly length / mass / tension force of its extradosed cable
- Environmental condition (Industrial, terrestrial or marine etc..)

Factors affecting durability must be analysed very thoroughly at the initial design stage or when extradosed cable system is going through the qualification process.

For various categories of exposure class, ranges from inner room environment to very aggressive industrial atmosphere / coastal, offshore in all conditions, multi-layer corrosion protection system as discussed earlier in this chapter satisfy, 100 years of design life of prestressing steel used in cables with high fatigue loading with present knowledge.

No scientific model is available, which can predict corrosion process over time as a function of exposure class or different level of fatigue. Hence, it is recommended to provide permanent multi-level corrosion protection to extradosed cable which is adequate for proven system of minimum 100 design life for all types of cable supported bridges [2].

Based on ease of accessibility and condition of its replaceability of mild steel components of anchorages and saddle, extradosed cable system supplier shall present the corrosion protection approach of his extradosed cable system in accordance with Table-8.2.

Table.8.2 : Corrosion protection philosophy for mild steel components of Anchorages & Saddle [2]

Aggressiveness of the environment (Very Severe Exposure Condition)				
Design Lifetime of the Cable Stay System	Accessibility or replaceability of the components	Durability of the corrosion protection system	Design Life of initially applied corrosion protection system	Period between subsequent maintenance operations
100 Years(1)	Replaceable	25 Years(1)	25 Years(2)	25 Years(2)
	Non replaceable Easy access	With maintenance > 100 years(1)	25 Years(2)	15 Years(2)
	Non-replaceable No access	100 years(1)	No Maintenance	

(1) Default value is 100 years (in case of higher value, it is to be specified by the designer/ client)

(2) Minimum value (actual value to be declared by the extradosed stay system supplier)

The extradosed cable system supplier shall define the type and thickness of coatings used on the mild steel cable components. He shall then demonstrate the durability of the mild steel components and declare in particular the required maintenance intervals and type of maintenance work anticipated such that the durability of the cable components is compatible with the design lifetime of the cables specified by the designer/client. He shall submit this information to the designer / client for approval.

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3. PTI DC45.1-18 Recommendation for stay cable design, testing and installation (Seventh Edition, November,2018).
4. Paper , Bridge Ice Accretion and De-and Anti-icing System :A Review by Kleissl Kenneth & Georgakis , Christos published in 7^{th} International Cable Supported Bridges Operators Conference-2010
5. IRC:24-Steel Road Bridges (Limit State Method)
6. EN 10337: Zinc and Zinc alloy coated prestressing steel wires and strands

CHAPTER - 9

STRUCTURAL MODELLING AND ANALYSIS

Structural modelling carried out for analysis must capture all aspects of behaviour of real structure under the acting loads. It is always to be kept in mind that analytical model will never be the exact representation of the actual structure, however, it should be close enough for practical purposes.

In general, linear elastic model is being adopted for structural analysis of extradosed bridges. Results obtained from linear elastic method multiplied by load factors are used to perform design checks for Serviceability limit state (SLS), Fatigue limit state (FLS) and Ultimate limit state (ULS).

For construction of extradosed bridges, various construction techniques are possible and the designer should choose whichever is best suited from site specific consideration. These are generally constructed using cantilever construction technique except for very short spans. Conventional form traveller (in case of in-situ construction) or lifting beams (in case of precast segmental construction), as used for the construction of cantilevered girder bridges and cable stayed bridges, are also being used for construction of extradosed bridges. Selection of superstructure types, span arrangement and material should be chosen at the design stage and should be consistent with the construction technique adopted.

The analysis of extradosed cables is similar to cable stayed bridges. Both are highly indeterminate structures with the stiffness of the deck, the cables, upper pylon and the piers affecting the distribution of the forces in the structure.

Analytical Models can be broadly grouped into two categories

(a) Global Analysis Model

(b) Local Analysis Model

9.1 Detailed Global Modelling of Extradosed bridges

Generally, a space frame model is sufficient for the detailed global analysis of extradosed bridges to work out forces (Bending moment, shear forces, torsion, stresses and displacements) in deck, piers, pylon and axial effects in stay cables due to vertical and horizontal in-plane loads. The structural analysis of the bridge should consider the changing structural systems during the course of construction, including varying internal effects and stresses at the construction stages. The analysis must also consider the redistribution of bending moments in the deck due to long term creep and shrinkage of the concrete.

This is achieved by modelling the deck girders as a linear beam elements, pier and pylons are modelled as linear or non-linear beam elements (depending on type of analysis being performed) and cables as a linear truss elements. Cables which are represented by truss elements are connected at lower end with Deck Girder and its upper end with pylon by rigid links (Refer Fig 9.1). The use of rigid links ensures that length and inclination angle of the cables in the model agrees with drawings. Since, centroid of cross-section of girder and stay cable anchoring point are at different levels, the same shall be taken care of, so that additional local moments induced in deck due to stressing of stays can be also taken into consideration. Similarly, rigid link connecting stay elements and pylon elements ensures that effect of stay forces get transferred to pylon.

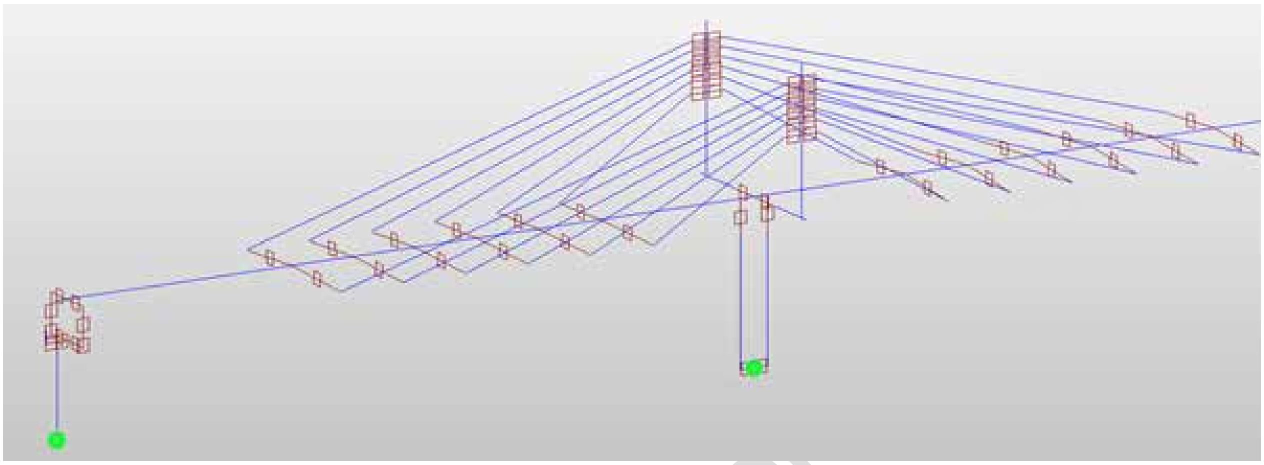


Fig-9.1 Analytical Fish Bone Model (Part) For Extradosed Bridge (for two planes of stay)

Superstructure model with two plane of stay (Fig 9.1) is called "fish bone model" where spine of box is represented by central beam elements and it is connected with pair of rigid links between girder elements and lower ends of Extradosed cables, representing pairs of ribs on either side of spine.

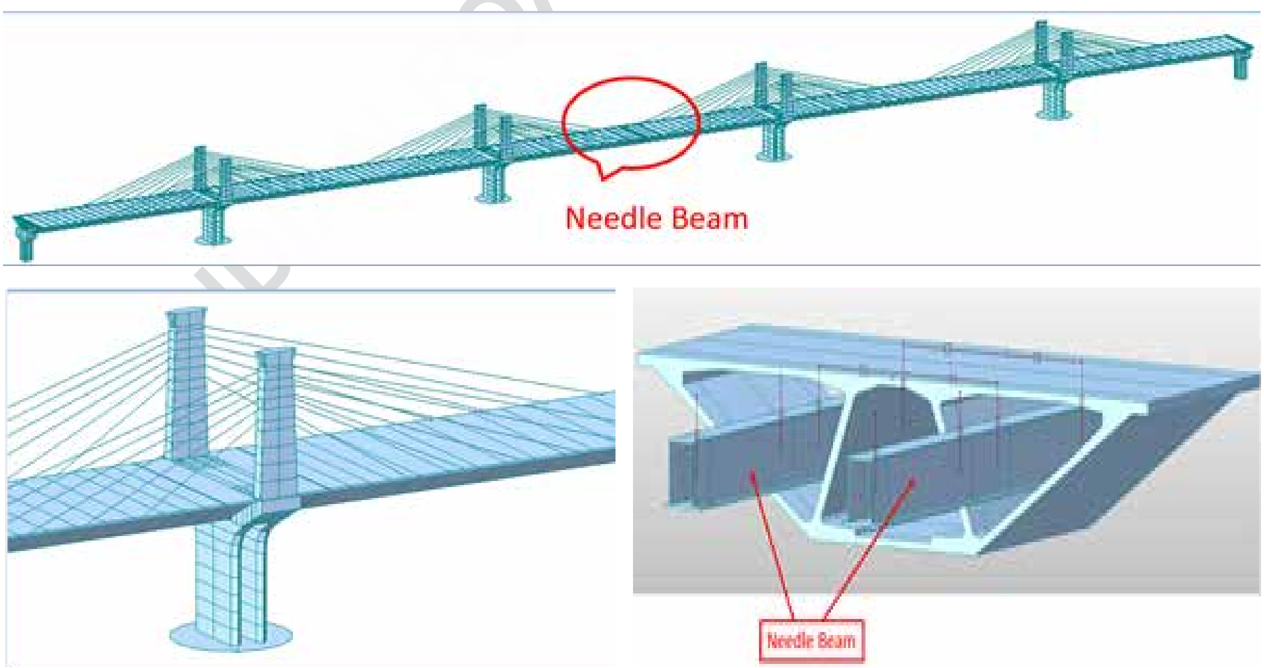


Fig-9.2 Solid Analytical Model View-Barapullah Extradosed Bridge Over River Yamuna, Delhi

Set of Bearing supporting the main girder at end piers are modelled with appropriate release in rotation / translation in both transverse & longitudinal directions of bridge.

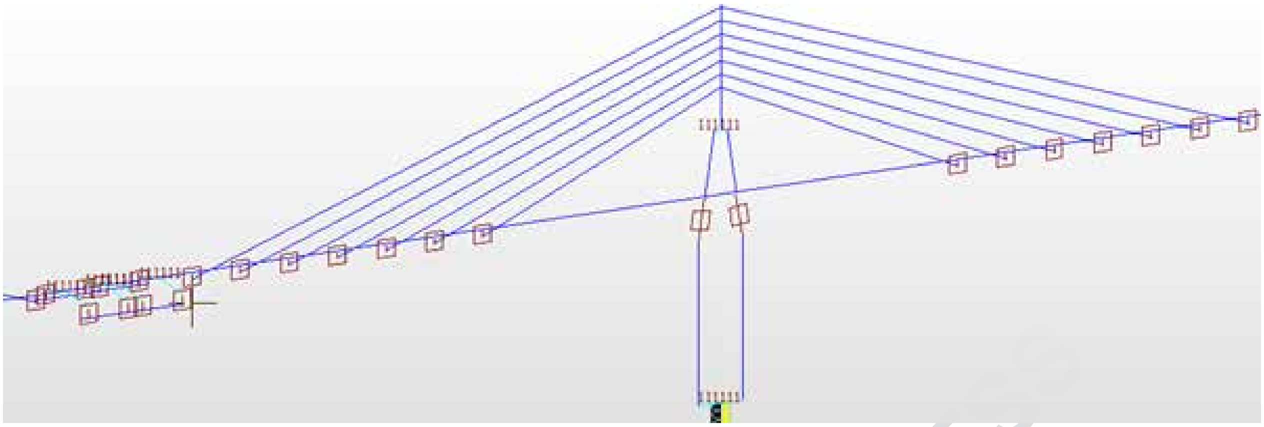


Fig-9.3 Analytical Model For Extradosed Bridge over River Ganga (Kacchi Dargah), Patna

Superstructure model with central, single plane of stay (Fig 9.3 & 9.4), where spine of box is represented by central beam elements and it is connected to lower ends of Extradosed cables with small rigid links, which represent the eccentricity between cable anchorage location and the top of superstructure deck level.

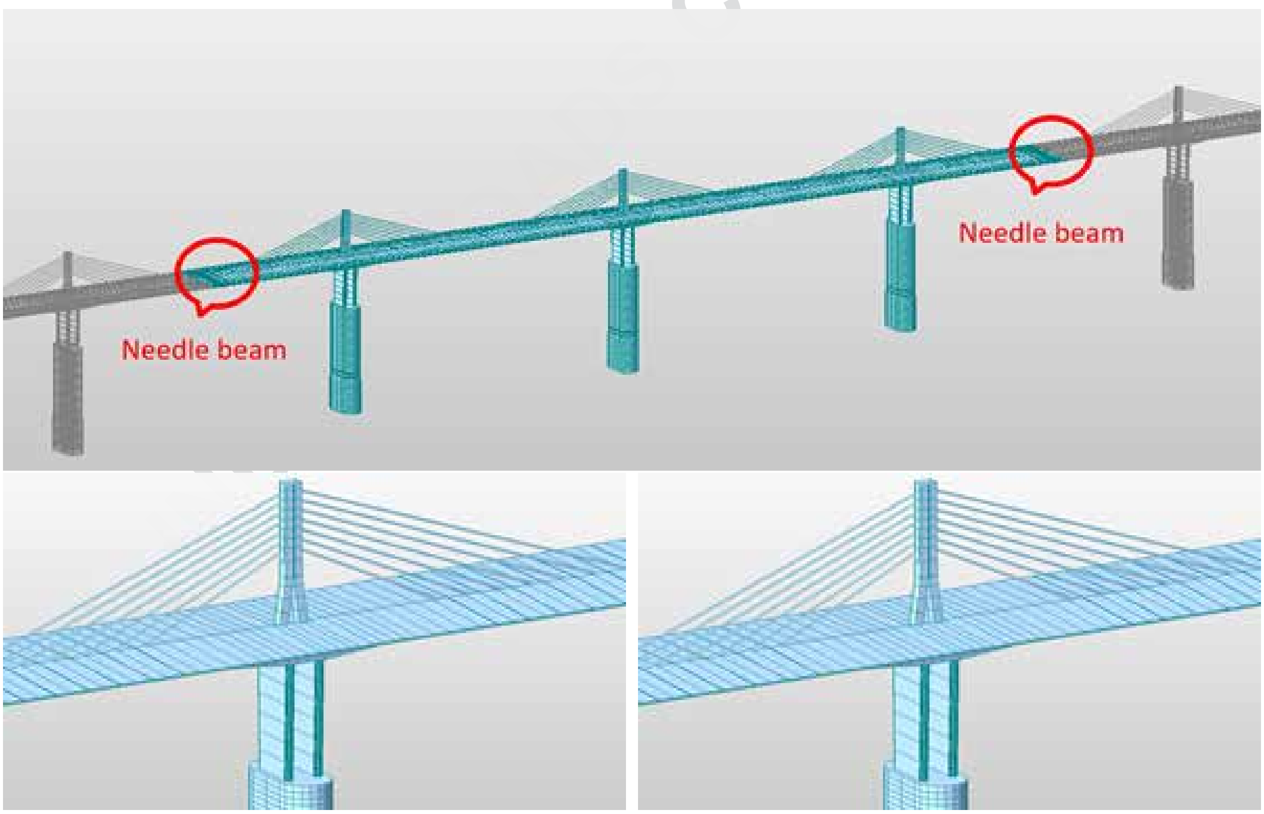


Fig 9.4 Solid Analytical Model View- Extradosed Bridge over River Ganga (Kacchi Dargah), Patna

Broadly, global analysis of extradosed bridges can be sub-divided into following parts:-

(a) Linear static analysis

(b) Linear elastic multi model response seismic analysis

9.1.1 *Linear Static Analysis*

Analytical model adopted for arriving at global results and software used should have following capabilities to analyse extradosed bridges:-

9.1.1.1 *Detailed Construction stage / Erection Load Effects*

(a) Ability to model the construction of the bridge as it is being planned to be installed.

(b) Activation and deactivation of elements with certain maturity.

(c) Capability to start the construction at several independent locations and at different times, each following its own schedule and finally to bring the component into an integrated, completed structure.

(d) Externally applied displacement to control deformation.

(e) Inclusion of stiffness of construction equipment, when the attachment of the equipment to the bridge components affects the stiffness of the bridge structure.

(f) Activation and deactivation of certain loading at certain stage of construction.

(g) Change in boundary condition of permanent or temporary supports.

(h) Addition and deletion of temporary members.

(i) Inclusion of spring supports to simulate temporary cast -in-situ supports

9.1.1.2 *Time Dependent Behaviour of Concrete Model*

(a) Full capability in modelling different concrete materials, based on model codes [3] or laboratory tests performed for the bridge under construction.

(b) Authentic time dependent modelling of creep and shrinkage of concrete of different maturities based on given type of cement used, relative humidity and notional thickness of structural elements

(c) Gain of compressive strength of concrete as a function of time

9.1.1.3 *Modelling for Prestressing tendons and its Time Dependent Behaviour*

(a) Capability of modelling of post tensioned bonded internal tendons and unbonded external tendons and mixed systems of both.

(b) Determination of losses in prestressing tendons due to friction, seating losses, stress relaxation, creep, shrinkage.

(c) Determination of primary and secondary effects (axial forces, bending moment, shear, torsion & displacements) of prestress.

9.1.1.4 *Modelling For Soil-Foundation Interaction*

(a) Capability of modelling of soil -foundation interaction is also one of the key aspect of modelling. Depending on foundation chosen and type of sub-strata on which foundation is rested, soil -foundation interaction between the two is required to be modelled as rotational / translation linear or non-linear springs.

9.1.1.5 *Modelling of Stay Cables & application of Forces in stay cables*

(a) Modelling of Extradosed cable

Stay cables are always modelled as truss elements. It is well known fact that cables can take only tension and not compression. But modelling stay cables as only tension truss element is not necessary.

Generally, Loads are applied during construction stage besides static loads (individual) applied during service stage (post-construction stage). There may be some load effects which can relieve the tension in stay cables. However, as long as cables retain some tension through all service stages, no separate 'tension only' analysis will be needed", and, stay cables shall be modelled as truss elements which can take both tension as well as compression, and eventually only tension after superimposing all the load cases.

Best example for understanding this aspect is a moving vehicular load analysis which is always performed in post-construction stages, and, which can cause tension or compression in stay forces depending on vehicular position as an isolated case. Additional tensile forces will increase the existing tension forces in stay cables and introduction of compression in stay will only result in reduction of existing tensile forces in stay.

(b) Modelling properties of stay cables

While modelling any extradosed cable, effect of geometric non-linearity due to cable sag effect needs to be investigated.

Hence, to account for cable sag, cable stays are usually modelled as straight chord members with an equivalent modulus of elasticity of cable.

Cable sag is a function of following:-

(i) Horizontal projected length of cable

(ii) Weight of cable

(iii) Tensile stress in cable

Equivalent modulus of elasticity of cable can be worked out by assuming the catenary of a cable with low sag/ chord length as given below:-

$$E_{eq} = \frac{E}{1 + \frac{(L_0 \gamma)^2}{12 f^3} E}$$

E_{eq} is the equivalent elastic modulus of inclined cables (KN/m^2);

E is the nominal elastic modulus of the cable (KN/m^2);

L_0 is horizontal projected length of the cable (m);

γ is the weight per unit volume of cable (87 KN/m^3 for strand inside HDPE sheathing injected with wax)

f is the permanent tensile stress in the cable (MPa).

Hence, for a cable length of 70m (would be longest cable) for a 180m span of extradosed bridge with a permanent stress of nearly 550 MPa (0.3 f_{pu}), for which there is very little sag, the equivalent modulus of elasticity of cables is only 0.5% lower than effective modulus of elasticity (refer Fig-9.5) which can be neglected for practical purposes for analysis of extradosed structures.

Thus, in the modelling of extradosed bridges, the cables can adequately and safely be modelled as linear truss elements without considering cable sag effects.

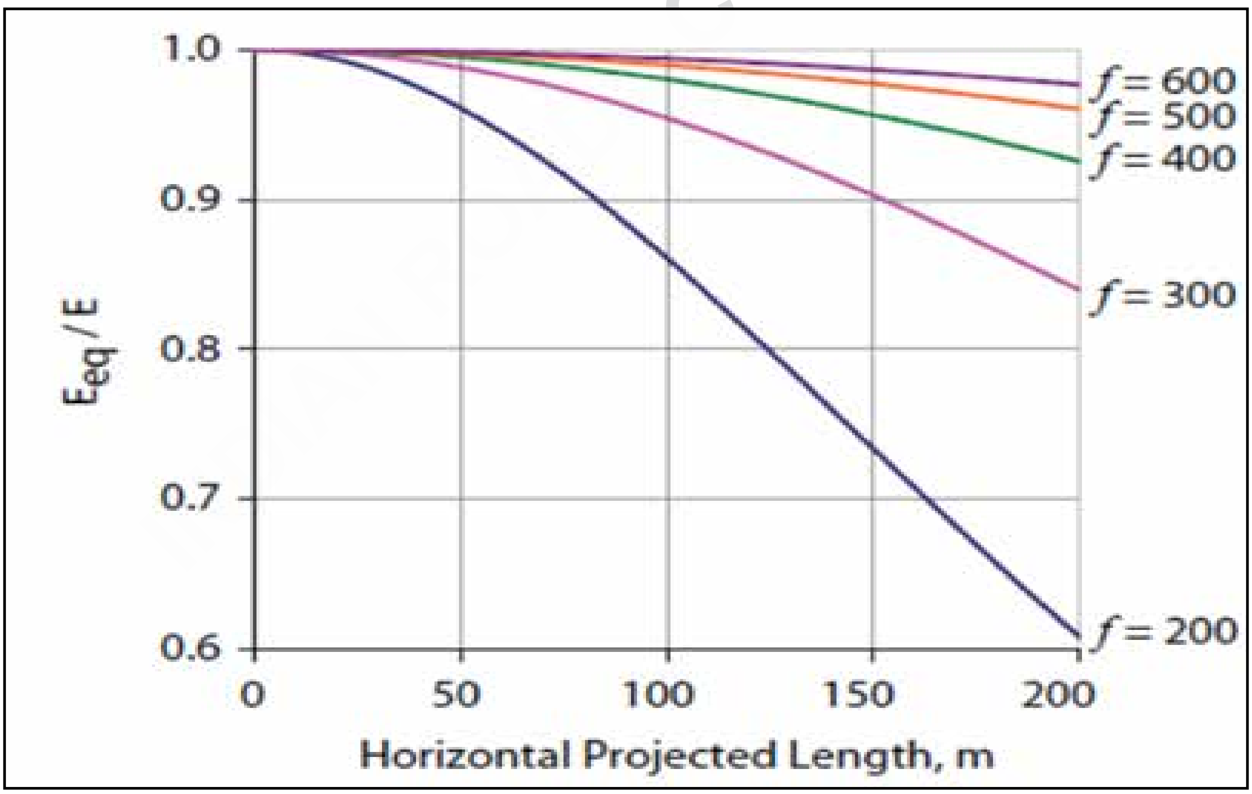


Fig-9.5 Ratio of Equivalent to Initial Modulus Of elasticity – Influence of a cable sag on its stiffness [1]

(c) Application of stay tension forces during construction

During construction, tension in stay cables is progressively increased which can be broadly classified in to three types:

(i) Initial tension forces in stay cable immediately applied after its erection during cantilever construction

(ii) Additional tension forces in stay cable in intermediate stages of construction

(iii) Additional tension forces in stay cables before final construction

Such multiple progressive tensioning of stay cables is required to control the stresses in girder to their limiting value and also to control displacements during cantilever construction.

Generally in softwares, initial and additional tensile forces introduced in stay cables during construction can be increased in two ways:-

(i) By adding additional pretension force in stay over and above existing force in stay. Hence tension forces in the stay keep on increasing (accumulating).

(ii) By replacing the force in the stay by higher tension force, by way of input. Hence whatever may the force in stay cable in previous stage is replaced by new tension force which will be locked with the newly defined input of increased force after deformation of structure.

Generally, latter option is being opted for analysis of extradosed bridges.

(iv) Modelling for Loss (breakage) of Extradosed cables

Effect of accidental loss (breakage) of one extradosed cable at any given time is also required to be analysed. Such breakage shall be represented by not only exerting forces in both anchorages in the direction opposite to cable stay tension but also by multiplying appropriate dynamic amplification factor. This will result in

(i) Re-distribution of tensile forces in rest of stay cables whose strength check is required to be made

(ii) Redistribution of forces (BM & shear forces) in supported girder whose strength and stability check is also required to be done.

(iii) Redistribution of reaction at supports

Hence Model should have capability to install or remove any stay cable at any stage also.

Breakage of extradosed cable is represented by a force exerted at both anchorages in the direction opposite to cable tension and weighted by a dynamic amplification factor. Since extradosed cables are made up of independent main tensile elements (PSS , PWS), It is unlikely that all the main tensile elements of cable would break simultaneously, the amplification factor is recommended to be 1.5 [6].

A loading diagram of accidental (breakage) loss of cable is shown in Fig 9.6 wherein a extradosed cable is anchored at top and bottom by anchorages (refer type-A, B and C of Fig 8.5 of chapter-8). Hence such force shall be applied at both the top and bottom anchorage locations in direction opposite to the cable tension. Such an exercise to study the effect of cable breakage needs to be performed for all cables one by one to get the worst effect in all other cables.

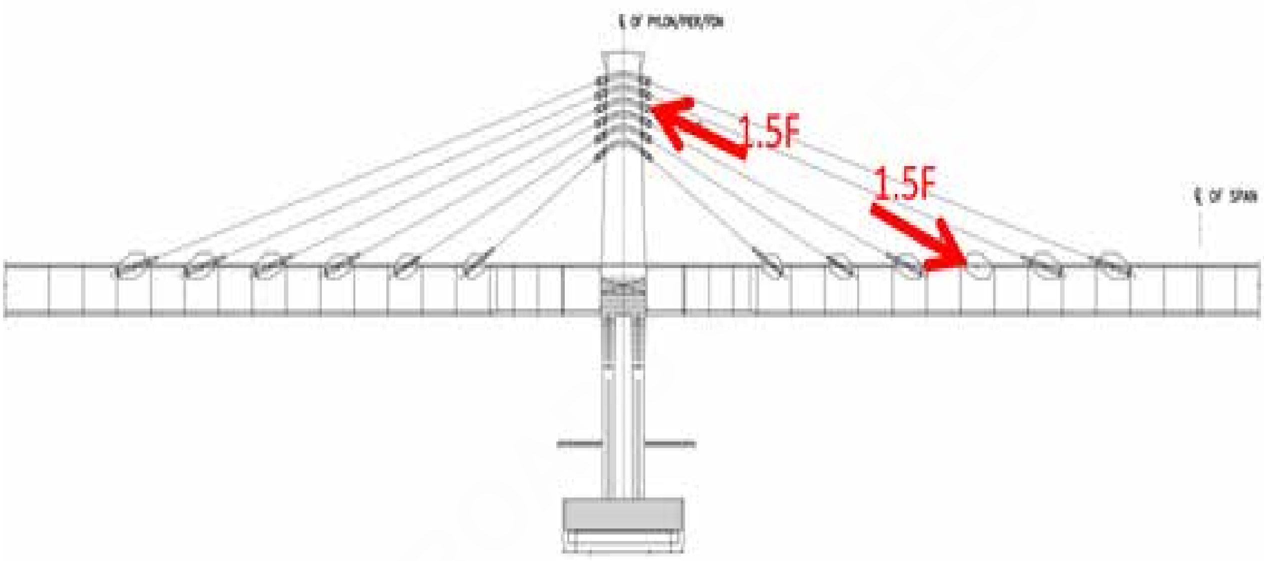
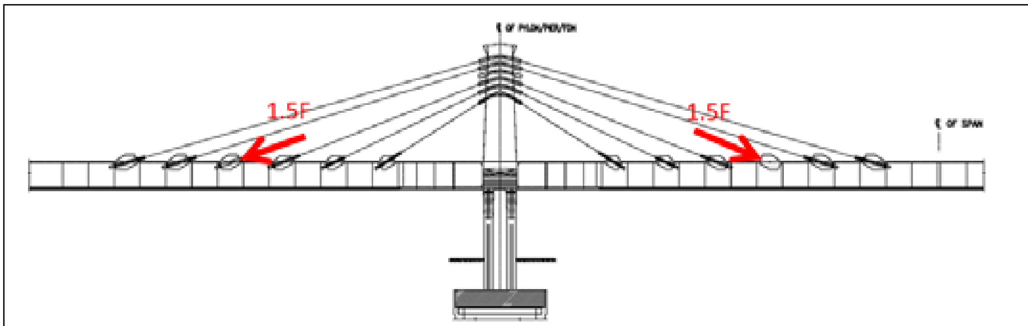
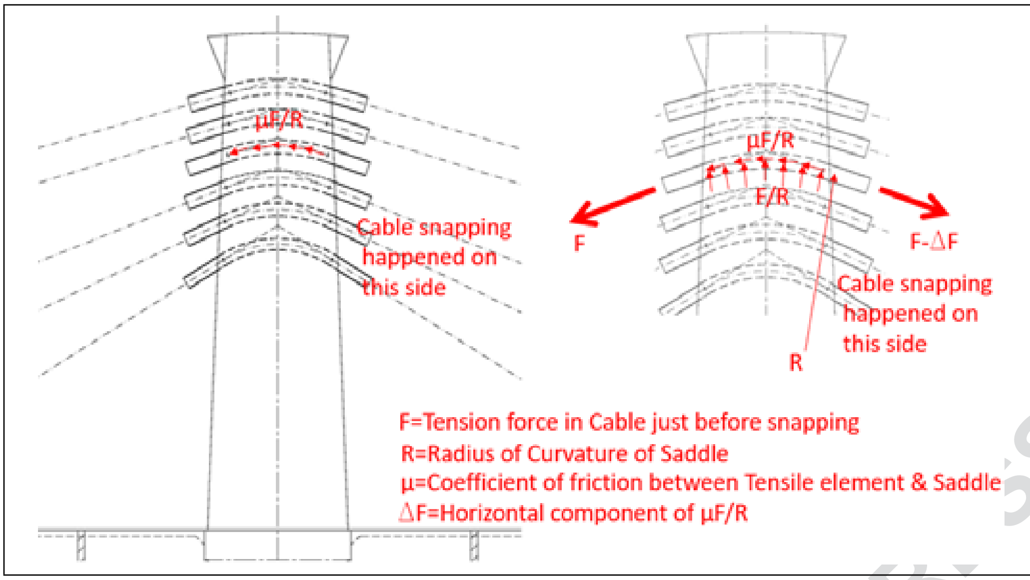


Fig 9.6 Accidental Loss (Breakage) Of Extradosed Cable (Where each cable is anchored at top & bottom by anchorage)

However, in case of pylon saddle where each strand of extradosed cables (refer type-D of chapter-8 (Section 8.11)) passes, in continuity, through individual tubes / holes within saddle body, combined frictional force generated due to radial action of axial tension in individual strand / cable shall be taken in opposite direction as an upper bound level of action effect due to accidental breakage of cable, as is shown in Fig 9.7.

	Forces Effect on Super-structure



Forces Effect on Pylon

Fig 9.7 Accidental Loss (Breakage) Of Extradosed Cable (Where each strand of cable is continuous passing over saddle in a individual hole)

9.1.1.6 *Modelling For Performing Effect of Vehicular Live Load*

In order to carry out moving load analysis, configuration of vehicle loads, impact factor, traffic lanes, Multi-Lane factors, vehicular load combinations [4] are required to be inputted to arrive at the envelope of maximum and minimum desired effect (BM or shear force or torsion) and other concurrent effects for all structural elements.

Fig 9.8 shows the Influence Line diagram and vehicular axle position for maximum sagging moment in center of penultimate span of a 5 spans extradosed bridge. Fig 9.9 shows Maximum Sagging / Maximum Hogging BM envelope for Superstructure for half bridge.

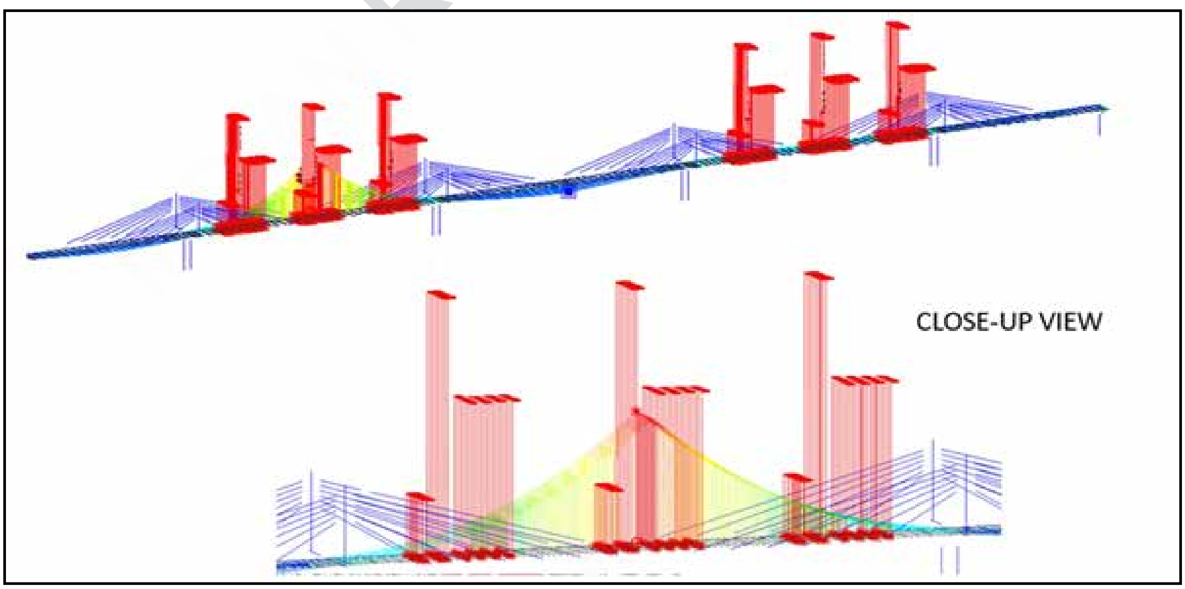


Fig-9.8 Graphical Representation of ILD Diagram For Max Sag BM in Penultimate Span and Axle Position on Model To achieve it

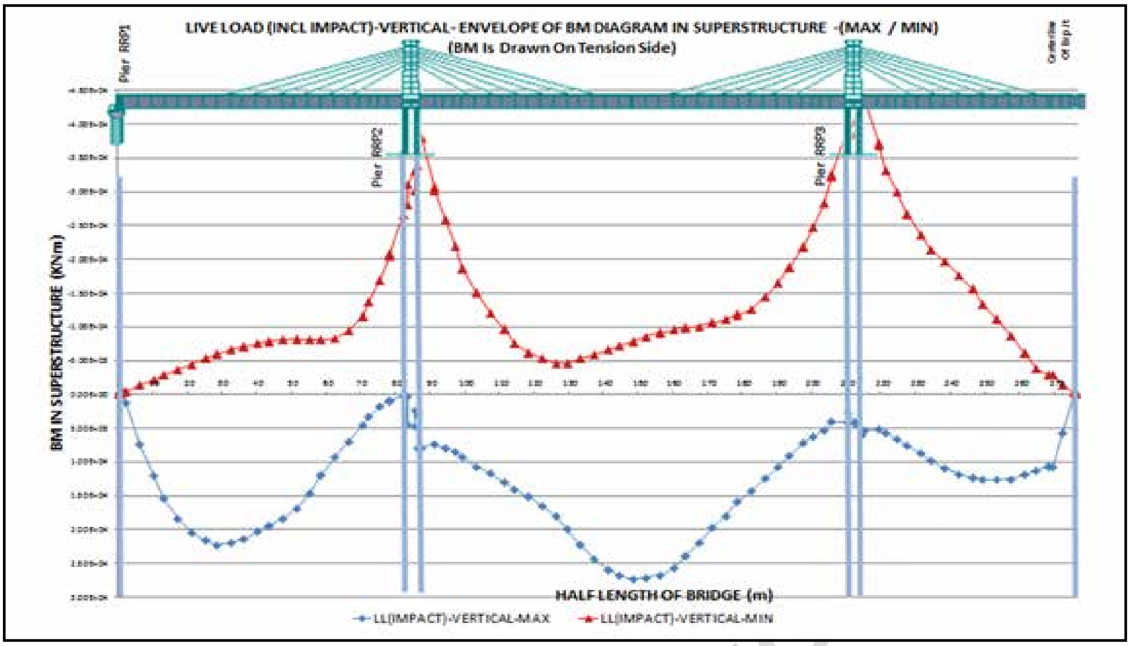


Fig 9.9 BM Envelope Due To Vehicular Loading

9.1.1.7 *Modelling for performing Settlement Effects of foundation*

If required, to account for differential settlements of foundation, model should be able to arrive at envelope of maximum and minimum desired force (BM or shear force or torsion) and other concurrent forces for all structural elements due to settlement of support or group of supports for defined magnitude of settlement.

9.1.1.8 *Thermal Stress Analysis*

Model should be able to analyse thermal effects such as,

(a) Global temperature fall and rise of superstructure deck

(b) Non-linear thermal gradient across the depth of structure

(c) Linear differential temperature across the depth of pylon.

(d) Differential temperature between cable stay and superstructure girder

9.1.2 *Linear Dynamic (Seismic) Analysis*

Extradosed bridges have a large variety in terms of structural system, connection between superstructure and substructure, number of planes and arrangement of cables, arrangement of bearings, and pier configurations. The seismic behaviour and hence design parameters will depend on the structural configuration.

A 3D dynamic analysis, including superstructure and substructure together, will be required, using response spectrum or time history method. In case of single plane of cables, the superstructure may be represented by a single spine. In case of multiple planes of cables, 3D modelling of superstructure, considering torsional mode of vibration, will be required.

In Linear elastic multi-modal response spectrum analysis, mode shapes and their corresponding natural frequency are determined. In a given mode, the spectral value corresponding to the calculated natural period is searched from the input of spectral data through linear interpolation. It is therefore recommended that spectral data at closer increments of natural periods be provided at the location of curvature changes. The range of natural periods for spectral data must be extended sufficiently to include the maximum and minimum natural periods obtained from eigenvalue analysis. Total response is determined by combining responses in various modes by mode combination procedure such as Square root of the sum of the squares (SRSS) or Complete Quadratic Combination (CQC) method. CQC method is suitable when the natural frequencies are close to each other, which is quite common in extradosed bridges. The number of modes should be sufficient to ensure 90% of mass participation. The ground motion effects should be included for all three principal directions of the bridge. The 100:30:30 combination rule (as given in Cl. 219.4 of IRC:SP:114) can be used for combining the member actions obtained from response spectrum analysis.

Elastic design forces considering response reduction factor equal to unity shall be considered for working out design forces for upper pylon (part to which stay cables are connected), superstructure and bearings or as transferred using capacity design principle. The response reduction factor for pier will depend on the configuration of the piers and their connection with the deck and the pylon, according to code IRC:SP:114. In case of piers with well-defined locations of plastic hinges, integral or bearing supported deck with piers, Response Reduction factor as defined in code IRC:SP:114 can be used. In case of squat (aspect (height to length) ratio less than 2) piers, and wall type piers (one dimension being much larger than the other), Response Reduction factor, R=1.0 shall be used in the transverse direction. Also, in case of piers with complex configurations with inclined pylons or members, where location of flexure plastic hinges cannot be clearly identified, and in case of piers having excessive axial load (more than the Pu,bal, in the P-M Interaction curve), where ductile behaviour of the structure cannot be achieved, Response Reduction Factor, R=1.0 shall be used.

The ductility provisions for seismic design are required to be made in various components of structure in accordance with relevant code of practice [2] & [3].

Further, Extradosed cables cannot take any compressive force and therefore, it should be ensured that the cables remain in tension (with minimum value as specified by system supplier) during the whole time-history of the earthquake. Extradosed cables may come into compression due to vertical vibrations of the deck caused by combined effect of horizontal and vertical ground motions. If this is indicated by analysis, design measures should be taken to ensure that the same is avoided and cables remain in tension.

Long span bridges are sensitive to spatial variation of ground motion, because of wave passage effect and variation in geotechnical conditions below the tower supports. The spatial variation of ground motion results in large pseudo-static forces in addition to the dynamic forces. Large forces and displacements may be induced in the structure on account of these factors. Therefore, multi-support response spectrum (MSRS) or multi-support ground motion analysis should be performed in case of bridges having distance, between farthest supports interconnected by a continuous superstructure, more than the limit as given in IRC:SP:114. The consideration of spatial variation is even more important in case of bridges having piers/towers located on different types of soil strata.

Extradosed bridges may need large expansion joints to cater for large movements in earthquakes. At design basis earthquake (DBE), the damage to expansion joint should not hamper movement of vehicles. Hence expansion joint shall be designed for DBR condition with appropriate value of response reduction factor to be used for working out displacement.

9.2 Detailed Local Modelling of Extradosed bridges

9.2.1 *Connection Detail Of Stay Cable With Superstructure*

Stay cable and Superstructure connection are subjected to very high local stresses. It can be tensile stress or compressive stress depending on which part of superstructure is being considered. For working out local effects of stay cable forces on deck, finite elements models are analysed to evaluate the local stresses in structural elements wherein cable stay forces are imparted. Such model may use plate and shell elements or brick elements depending on size and shape of elements on which forces are imparted.

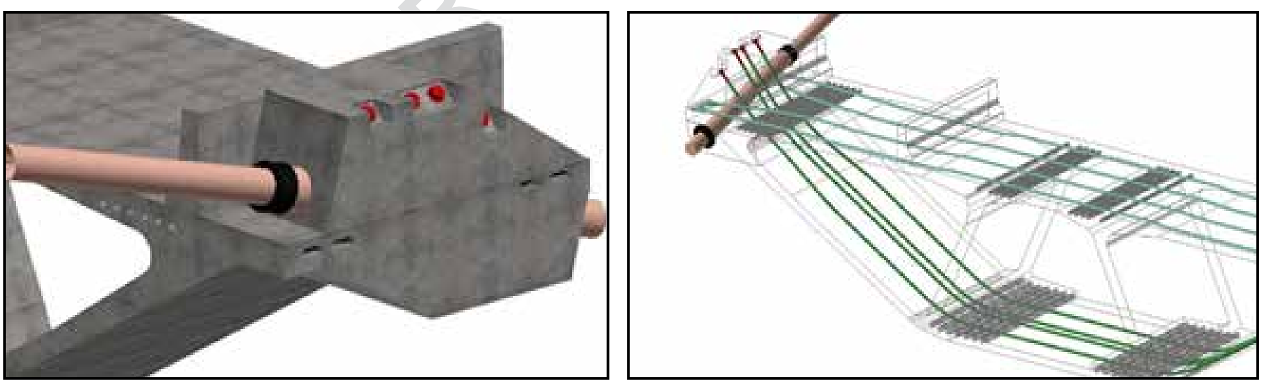


Fig-9.10 Connection detail of Stay & Superstructure (For laterally supported extradosed Bridge)- Barapullah Extradosed Bridge Over River Yamuna,Delhi

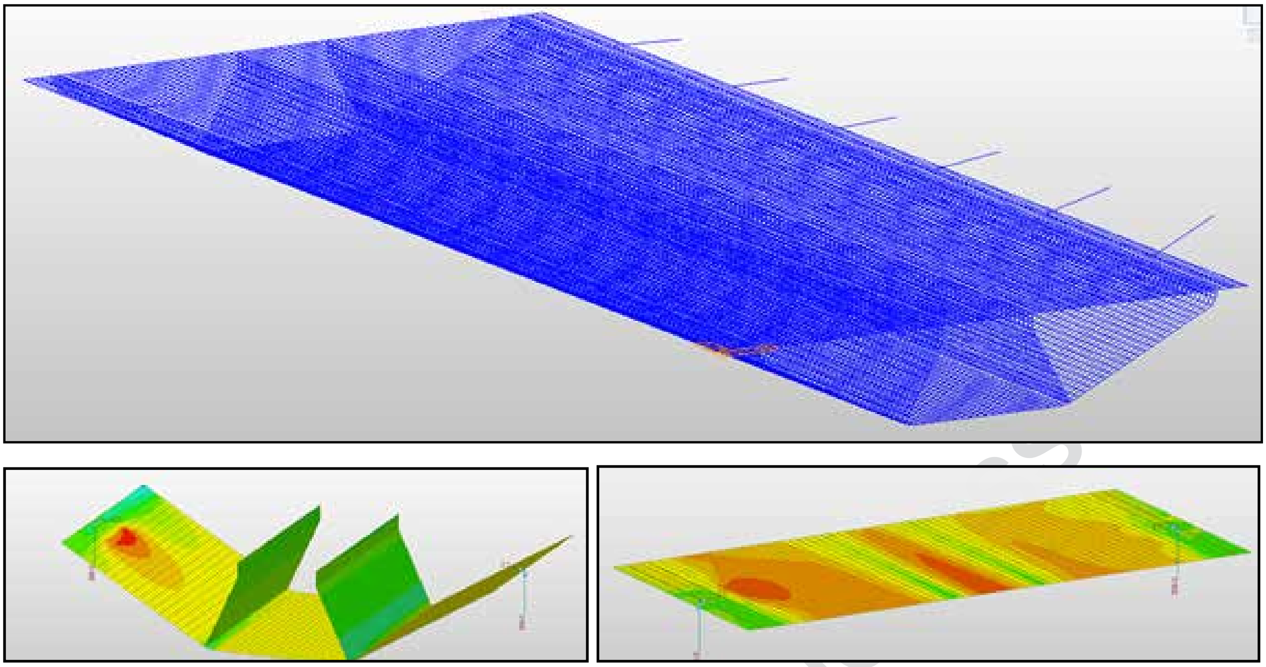


Fig 9.11 Analytical Model For Superstructure (top) , Local stresses in Web (Left bottom) and Deck Slab((Right Bottom)- Barapullah Extradosed Bridge Over River Yamuna,Delhi

Fig-9.10 shows a connection detail of laterally unsupported superstructure deck connected by extradosed stay cable at its extreme ends at deck level. For analysing the connection zone of stay cable with superstructure, one needs to analyse a model of complete superstructure box of finite length discretised into plate shell elements and imparted with stay forces. Such connection induces very high local tensile stresses in webs at superstructure-deck connection (refer Fig-9.11) and Compressive Stresses in deck slab. Such tensile stresses in webs are being negated by very high compressive stresses by providing localized webs cables (refer Fig 9.10).

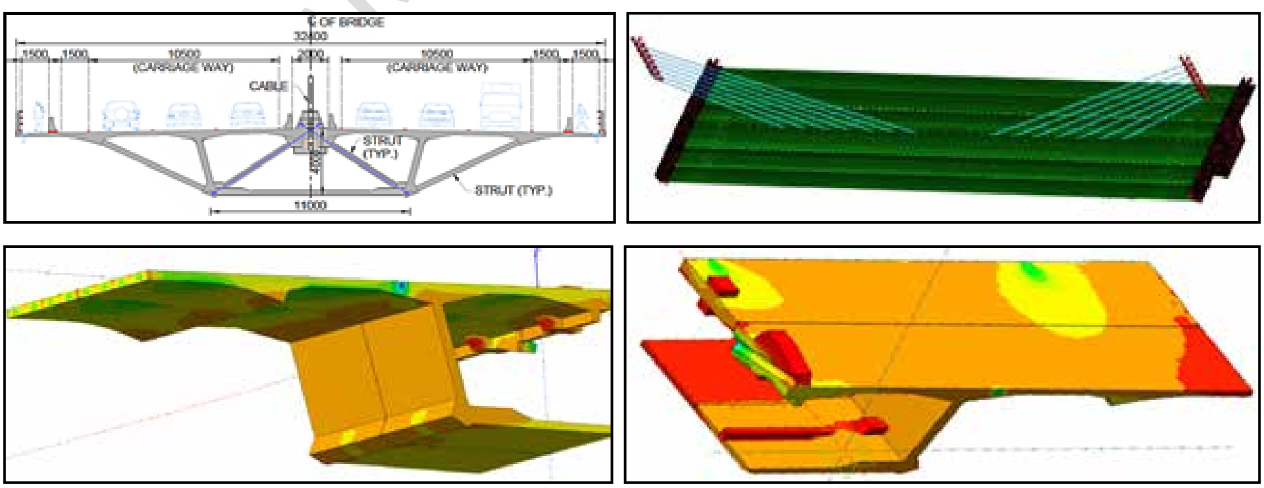


Fig 9.12 Analytical Model For Superstructure (top), Local stresses in Inner strut fixing block (Left bottom) and Cable anchorage location (Right Bottom)- Extradosed Bridge over River Ganga (Kacchi Dargah), Patna

Fig-9.12 shows a model for connection detail of centrally supported superstructure deck connected by single plane of extradosed stay cables below deck level.

9.2.2 Connection Detail Of Stay Cable With Saddle

Saddle which is embedded in upper body of pylon anchors the stay cables on either of its two sides. (Refer Fig-9.13). Such saddles (Type-C as defined in chapter-8) are required to be analysed and designed for following forces in Serviceability limit state Ultimate Limit State, Fatigue Limit State & Accidental breakage of Extradosed cable:-.

(a) Stay Cable axial forces

(b) Stay transversal forces

(c) Stay Cable unbalance forces in case of loss of cable event or in service condition

Fig 9.13 Upper Pylon with Saddle To Connect Stay Cable on both of its sides (Left), Fabricated saddle to anchor stay cable (Right)- Barapullah Extradosed Bridge Over River Yamuna

Saddle (Type-C) is to be analysed with solid finite element (using 8 noded brick element) modelling (refer Fig 9.14). All plates are modelled individually and connected using appropriate constraints. Stay forces shall be imparted as circular ring patch load on bearing plate of saddle taking account of eccentricity of 5mm in lateral direction.

Loading of stay forces corresponding to serviceable load shall be imparted and linear elastic model shall be used for analysis. It has to be ensured that principle tensile stresses are limited within yield stresses of material and deformation remains under control as per the requirement of stay system requirements.

Loading of stay forces corresponding to ultimate condition shall be imparted and non-linear analysis (non-linearity in material) shall be performed. For such purposes Vin-Mises model can be followed for such welded plated structures. It has to be ensured that principal strain in material are limited to 5.0% (max) [8] or as defined by Extradosed Cable system supplier. Only local partial plasticization of plated structure shall be permitted under ultimate condition as per good engineering practices with limiting stresses equal to ultimate stresses of material with partial safety factor under ultimate limit state.

Loading of stay forces corresponding to fatigue loading shall be imparted and it has to be ensured that fluctuating stresses in all welded connection types shall be able to with stand required cycles as per relevant code of practice [5].

Connection of such saddle with surrounding concrete (using welded headed studs) shall ensure transfer of differential forces on either side of saddle in service condition or accidental breakage of stay cable condition. Careful detailing of reinforcement around such saddle is required to transfer local forces to surrounding concrete

In all service conditions, radial pressure is transferred to concrete through the bearing at bottom of saddle. It shall be also ensured that bearing stresses in concrete underneath the saddle shall be limited to cl 16.11.1 of IRC:112 (without considering any relief due to dispersion as it is not possible in one direction). It is to be also noted that cumulation of transfer of such vertical stresses from one saddle to saddle below shall be also accounted for.

Vertical component of such radial forces underneath each saddle also causes transverse splitting tensile forces (due to dispersion in one direction) and such effect must be taken care of.

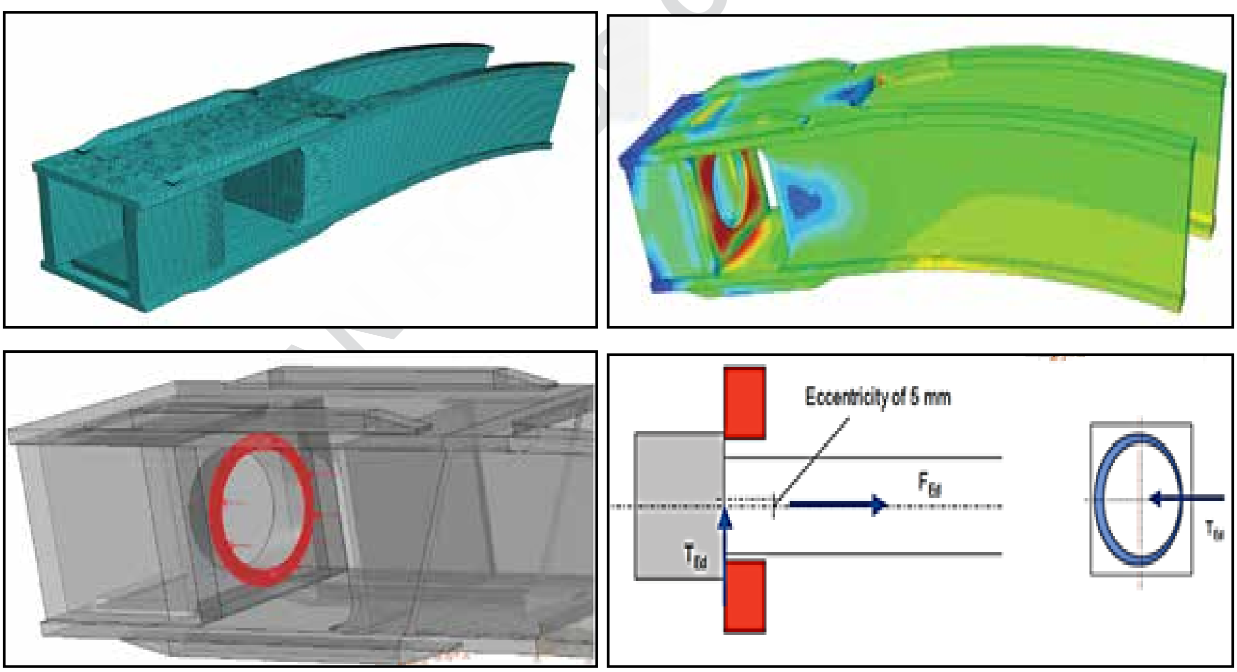


Fig-9.14 Solid finite element modelling of Saddle and Stay Forces Circular Ring eccentric loading (Left) and Stress Contours (right)

To ensure the transfer and diffusion of unbalanced axial forces in stay cables anchored on either side of saddle, mechanical connection in the form of shear studs are required to be provided on vertical outer body of saddle. Friction between saddle face and surrounding concrete shall not be considered for transfer of such forces.

9.3 Graphical Representation of Model and Analytical Results

Apart from the essential features listed in the preceding, there are number of convenience requirements such as ability to view the modelled structure, imparted loading and the computation solutions (bending moment, shear forces, axial forces, stresses, mode shapes, deflected shapes) in pre-construction stage as well as post-construction stage. Graphical representation of the same, greatly eliminates error in input data generation while modelling the structure and also helps in understanding and interpretation of analytical results.

9.4 Multi-Spans Extradosed Bridge

For very long bridges having extradosed bridge form, it is required to divide these into number of sub-extradosed units between expansion joints. Position of such expansion joints in superstructure determines the span configurations of sub-extradosed units. There are many alternatives for multi-span extradosed bridges as enumerated below:-

9.4.1 *Multi-Span Extradosed Bridge Units with Unequal Intermediate Spans (having deck expansion joint at piers)*

In case, it is intended to provide expansion joint in superstructure at pier top, shorter end span (equal to 60% to 65% of main span) is required for each extradosed bridge sub unit. This will result in unequal variation of all intermediate spans of extradosed bridge (refer Fig 9.15).

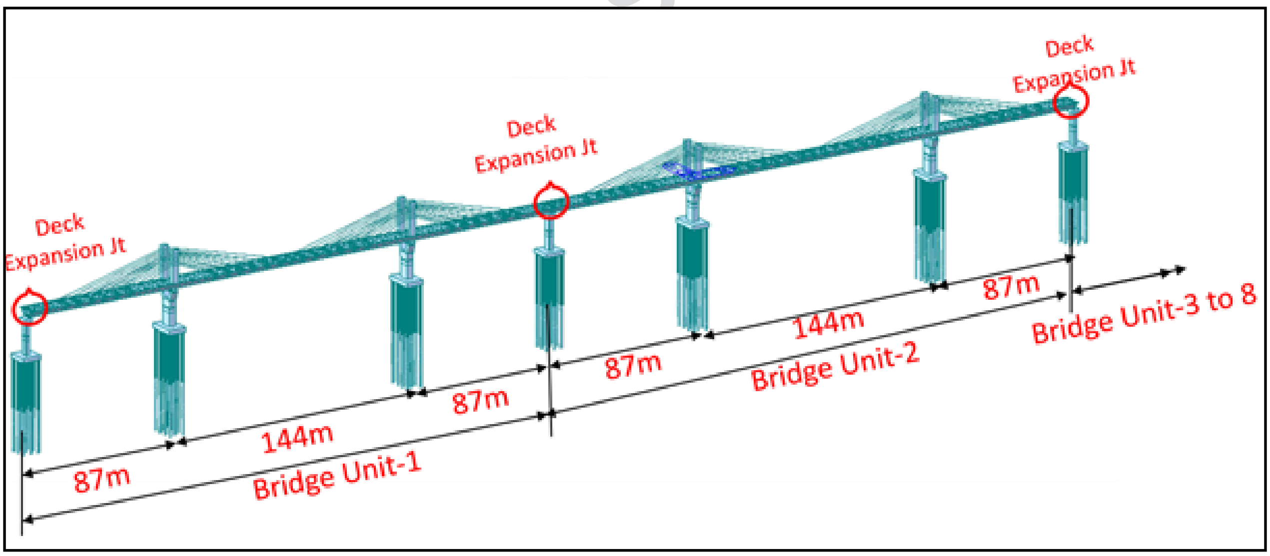


Fig-9.15 Multi-Span Extradosed Bridge (With All Unequal Intermediate Spans with deck expansion joint at intermediate piers)-Balia Bridge over river Ganga, UP-Bihar Border

9.4.2 Multi-Span Extradosed Bridge Units with Equal Intermediate Spans (having deck expansion joint at center of span)

Another option is to provide all intermediate spans of equal span length. In such case, expansion joint in the superstructure is provided at the middle of every alternate span, thus forming a Pi-shaped superstructure/substructure having a expansion joint at each end of the bridge (refer Fig-9.16).

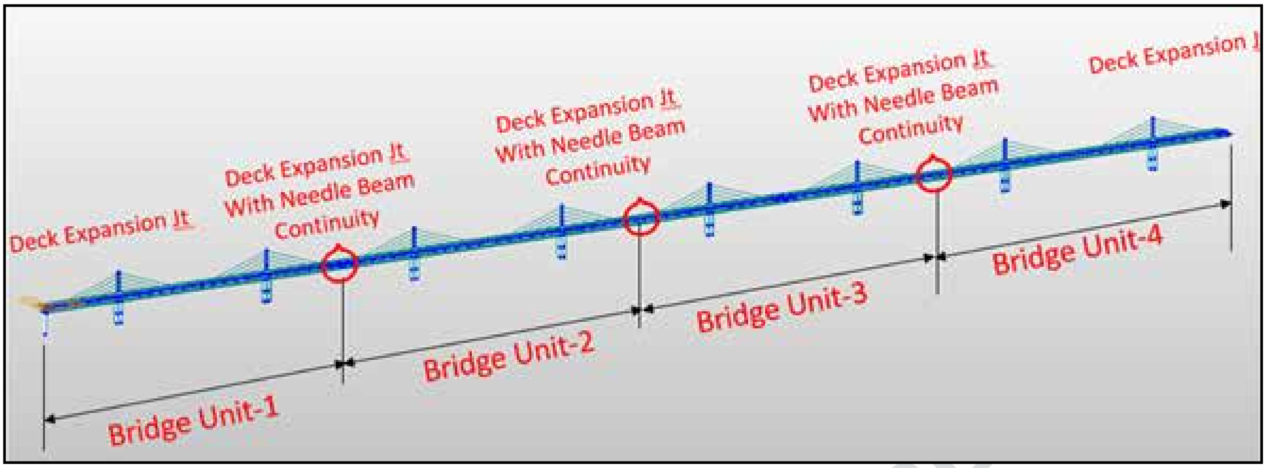


Fig-9.16 Multi-Span Extradosed Bridge (With All Equal Intermediate Spans having deck expansion joint in middle of span) -Arrah Chabra Bridge over river Ganga, Bihar

Such two adjoining superstructure bridge units with expansion joint located in center of span are being connected by a pair of needle beam and a set of bearings connecting as shown in Fig-9.2, Fig 9.17 & Fig 9.18.

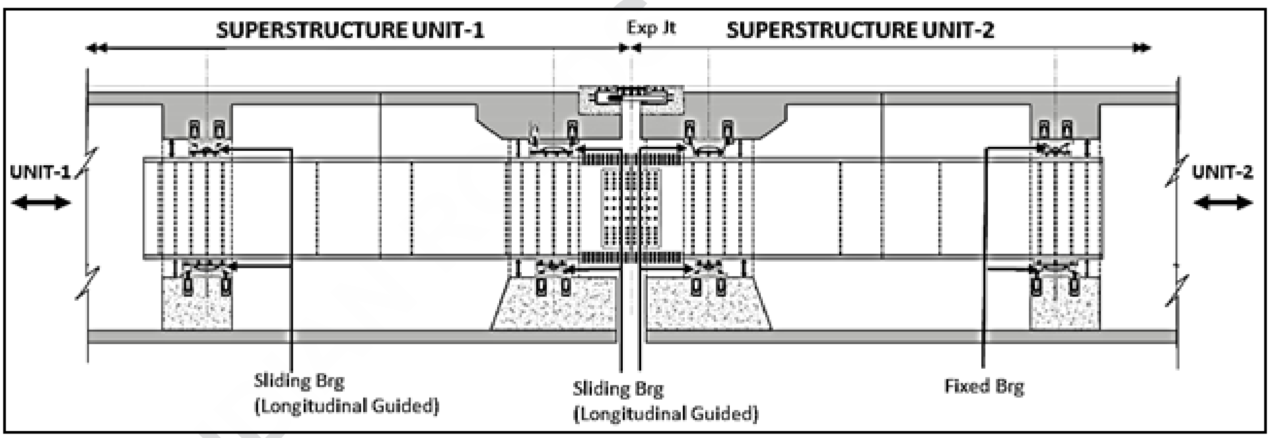


Fig 9.17 Needle Beam Arrangement at center of span-Barapullah Extradosed Bridge, Delhi

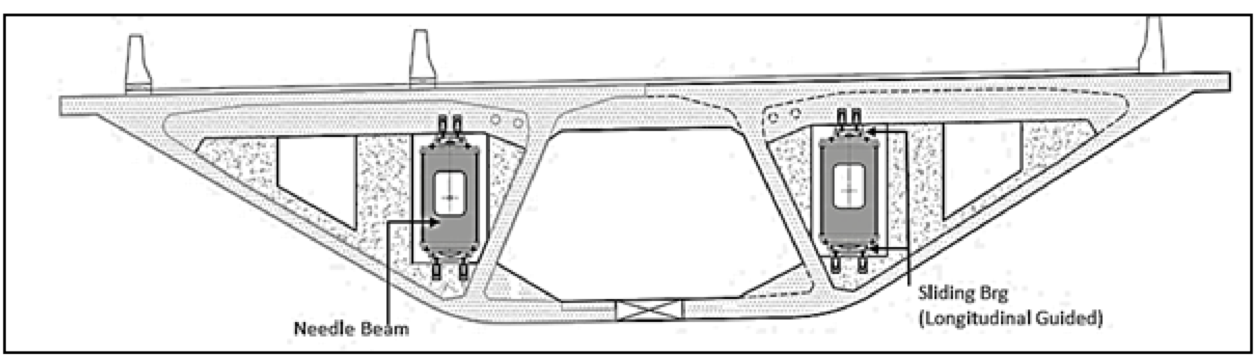


Fig 9.18 Needle beam Inside Superstructure Cross-Section

Pair of needle beams is in the form of twin rectangular shaped stiff box beams, which are introduced inside of superstructure of both adjoining box sections and supported on four cross diaphragms. Each needle beam is connected to both adjoining superstructure units with bearings at top and bottom and at two points, thus eight bearings are required per needle beam. Bearings are provided at top & bottom in order to take care of reversal of loads that can happen during seismic event. At each bearing point, a stiff cross diaphragm in superstructure ensures local transfer of vertical and lateral shear from needle beams to webs of superstructure box section.

While detailing the needle beam-superstructure connection by sets of bearings, it shall be ensured that each bearing can be easily inspected from all sides and if required (in future), it can be easily replaced.

Main objective of pair of needle beams at expansion joints is to provide shear transfer and moment resistance in addition to controlling differential deflection (and ensuring vertical and lateral compatibility) at the cantilever end (at center of span) of each superstructure frame unit. At such location, needle beam and sets of bearings provided ensure same longitudinal slope and lateral slope at cantilever tip of both adjoining superstructure units. Translation restraint in various bearings are provided in such a manner that it allows free longitudinal expansion / contraction movement of superstructure due to temperature variation / creep and shrinkage of concrete. It also allows relative longitudinal movement between two adjoining extradosed frame units in case of seismic event or in case of application of longitudinal braking / tractive forces.

In order to analyse, such a beam and its behaviour with both adjoining superstructure using bearing, rigid links are required connecting beam elements representing superstructure and beam elements representing needle beam (refer Fig 9.2). End releases are required to be given to each such rigid link representing bearing translation restraint which is provided at each specific bearing provided in each needle beam - superstructure connection location. Such rigid link ensures rotation of needle beam as same as superstructure but it allows the needle beam longitudinal translation with respect to one of the adjoining superstructure unit.

9.4.3 *Multi-Span Extradosed Bridge Continuous Units having equal intermediate span (With Adoption Of Shock Transmission Unit)*

Another option is to provide all intermediate spans of equal span length and provide continuous extradosed bridge units from one end to another end of bridge. In such case, bearing arrangement shall be provided in such way that longitudinal translation restraint to superstructure is being provided at pier closest to centre of continuous unit and at all piers superstructure transverse translation restraint shall be achieved.

In addition to above, shock transmission units (STU) connecting superstructure shall be also provided to one or more piers which are adjacent to pier provided with longitudinal translation restraint provided by bearing. Number of piers on which STU are provided depends on various factor such as length of complete continuous unit, capacity of fixed pier and pier provided with STU to resist in-plane forces.

The STU consists of a machined cylinder with a transmission rod that is connected at one end to superstructure and at the other end to the piston inside the cylinder. The medium within the cylinder is a specially formulated silicon compound, precisely designed for the performance characteristic of a project.

Fig 9.19 Shock Transmission Unit

Such STU devices provide velocity dependent restraint to relative displacement between superstructure and supporting piers. By tuning the gap desired between the piston and the cylinder wall, different characteristics can be achieved.

For low velocity, superstructure translation such as those due to temperature effect, creep or shrinkage of the superstructure, the movement is practically free (with very minimal reaction-maximum up to 10% of design forces). In such case silicon is able to squeeze through the gap between the piston and cylinder wall.

For high velocity superstructure translation such as those due to seismic or braking actions, the movement of piston is blocked (locked) as the silicon compound cannot pass fast enough around the piston. Thus the device acts practically as rigid connection between the superstructure and substructure. In-plane inertial (seismic) forces along longitudinal direction of bridge are shared between piers (pier provided with longitudinal translation restraint with bearing and all other piers in which STU is provided) proportional to their individual flexural stiffness.

Such devices shall be provided with sufficient displacement capability for all slow velocity actions of superstructure and full force capacity at their maximum displaced status.

Distribution of forces between the various restraining devices (at fixed pier) & STU (provided in piers other than fixed piers) is not uniform as STU will not get activated simultaneously. This is because of the reasons given below: (please check)

- Tolerances within the structure
- Tolerances within stiffness of each device
- Tolerances within the lock-up velocity of each STU device
- Tolerances within the STU-bracket system

In case STU is designed without force limiting function, the difference in tolerances leads to considerably higher input forces on seismic devices & such devices need to be designed for 150% of the rated design force (reliability factor of shock transmitters on their design force F_d shall be γ_x=1.5 [7]). Moreover this increase in design force is not only applicable for the STU devices but for the support brackets, the anchor system and finally for the structure itself too.

Shock transmission with Load limiting function (STL) on the other hand displays the same working principle as an STU, but offers the possibility of a load limitation. One valve in each direction opens after reaching a predetermined pressure level: the device yields, causing a movement of the structure.

In case of (STL), value of the reliability factor can be reduced to γ_x =1.1 [7] and shall be applied to the design system force F_0 as specified by the designer. Usage of STL ensures that all shock transmitters in a structure activate together.

In such cases, STL are used to resist seismic forces which shall be lower of :-

- The reaction corresponding to the capacity design effects in case of ductile piers.
- The elastic seismic forces arrived after sharing of inertial longitudinal forces among different piers as discussed above.

All STU / STL shall be accessible for inspection and maintenance / replacement.

For modelling of STU / STL in structural analytical model, rigid link connecting superstructure and substructure are provided for short term loading such as for earthquake forces and braking / tractive in-plane forces of carriageway vehicles while for all slow acting load, it is modelled with elastic link with very minimal axial stiffness.

9.5 Camber Control Analysis

In all segmentally constructed bridge, one needs to cast the segments with camber corrections so that once all segments get erected /cast at site, one can achieve desired profile of bridge in its final shape.

Such pre-determined profile of segments with respect to adjacent match cast segment requires a detailed pre-camber control analysis, wherein all construction stages of erection of such prefabricated segments to build a complete bridge are modelled with accurate loading (temporary and permanent) activated at each construction stage (step) with most accurate predicted time line.

Time tag for each activity is very important so that time dependent effects of material used such as creep of concrete, shrinkage of concrete, relaxation effect of prestressing steel can be accurately accounted for. Such effects can cause deformation of each erected segment depending on stress level history in each segment during construction of bridge, age of concrete of segments at the time of its erection in bridge unit and age of concrete of segment at a construction stage.

Each construction step analysis requires calculations of

(a) Current step displacement (δ_{ab}) at each node:- It is displacement of nodes for selected construction stages due to activities performed within each construction stage (step)

(b) Virtual Displacement at each node:- which occurs at a node before the corresponding node is even activated.

(c) Accumulative / Net displacement nodes:- It is net displacement of nodes up to end of selected construction stage, i.e it is summation of current displacement of node which occurs after the node is activated up to selected construction stage.

(d) Real displacement of node:- It is summation of virtual displacements which occur before the corresponding node is even activated and net displacement, which occurs after the node is activated.

9.5.1 *Camber Control for Precast Segmentally Constructed Units*

A simple example of cantilever erection of three precast segments in three stages is enumerated in Fig 9.20. Current step displacement and virtual displacement at each node at construction stages are shown. Such displacement values results are extracted by performing stage analysis for each stage separately under corresponding stage loading.

Real displacement profile at the end of erection of each construction step is shown in last figure of Fig-9.20.

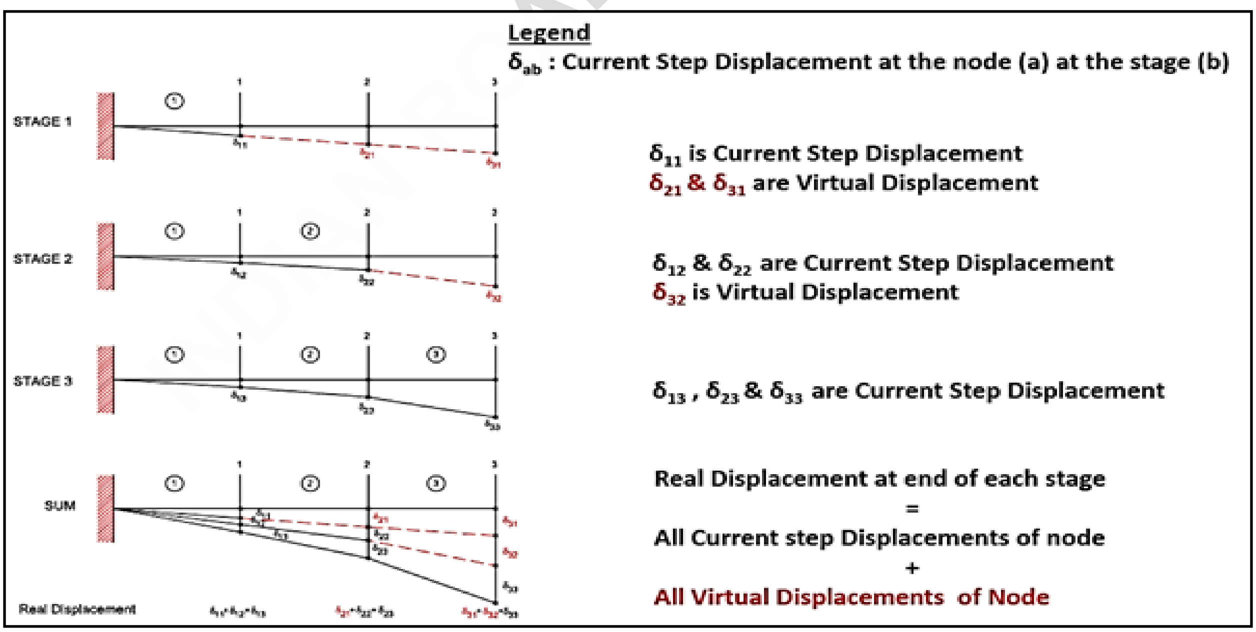


Fig 9.20 Real Displacement at Node (For Precast segments)

With no camber correction done, and, with camber correction done at each construction stage, profile of erected unit at end of each construction stage is shown in Fig 9.21.

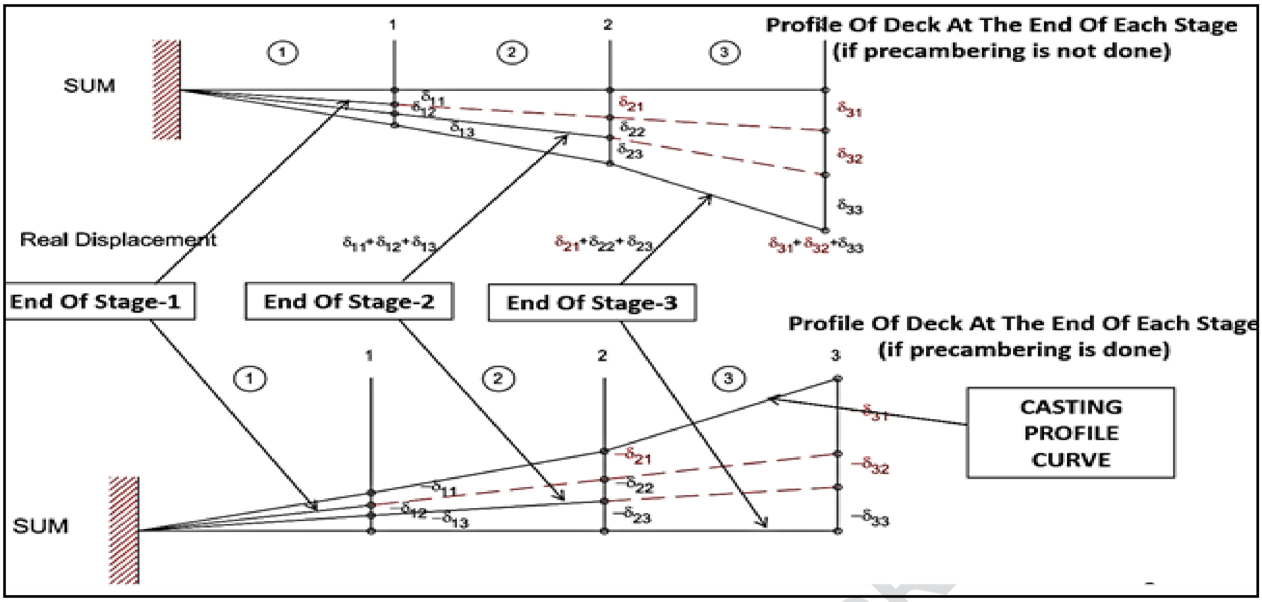


Fig 9.21 Precambering Using Real Displacement For Precast Segments

By reversing the final real displacement profile of erected unit at end of construction, one can work out casting profile curve of segments. To achieve a final structure with no deformation, the angle (ψ_i) between two segments can be worked out from real displacement data (refer Fig 9.22). Such calculated camber correction of angle (ψ_i) between match cast segment is required to be provided while casting of segment.

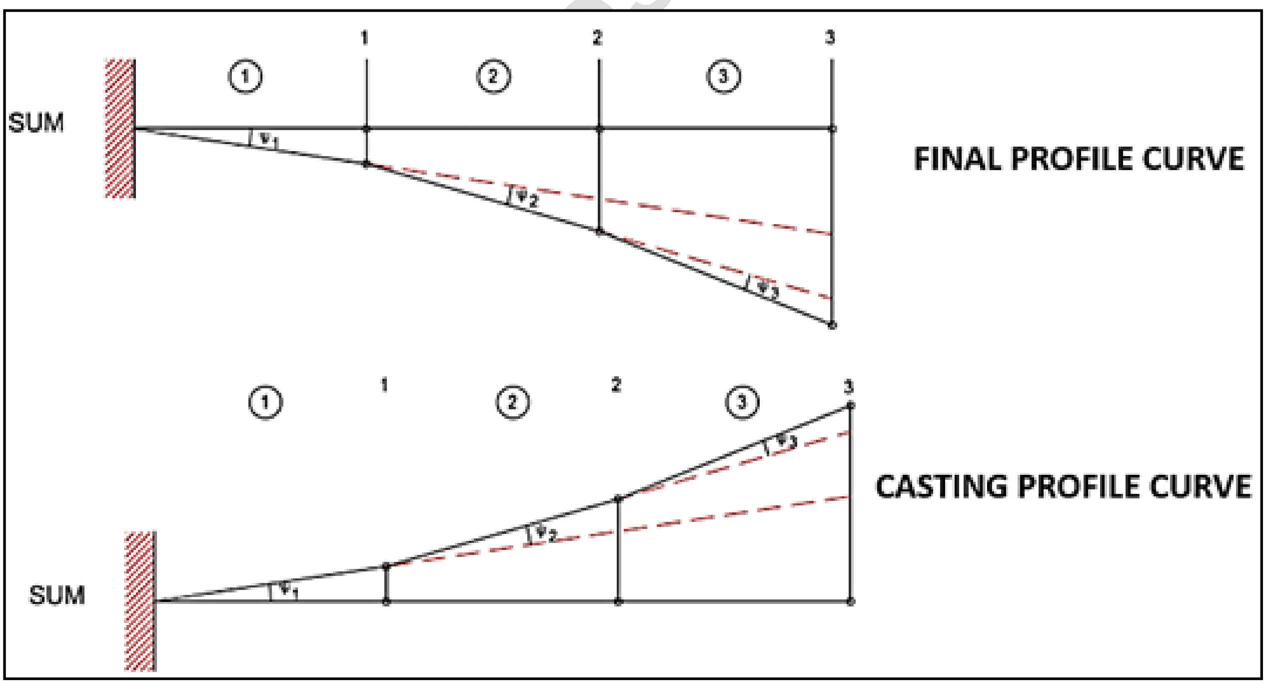


Fig 9.22 Casting curve Using Real Displacement -For Precast segments

9.5.2 *Camber Control for Cast-In-Situ Segmentally Constructed Units*

A simple example of cantilever erection of three cast-in-situ segments in three stages is enumerated in Fig 9.23. Current step displacement at each node at end of each construction stages are shown. Such displacement values results are extracted by performing stage analysis for each stage separately under corresponding stage loading.

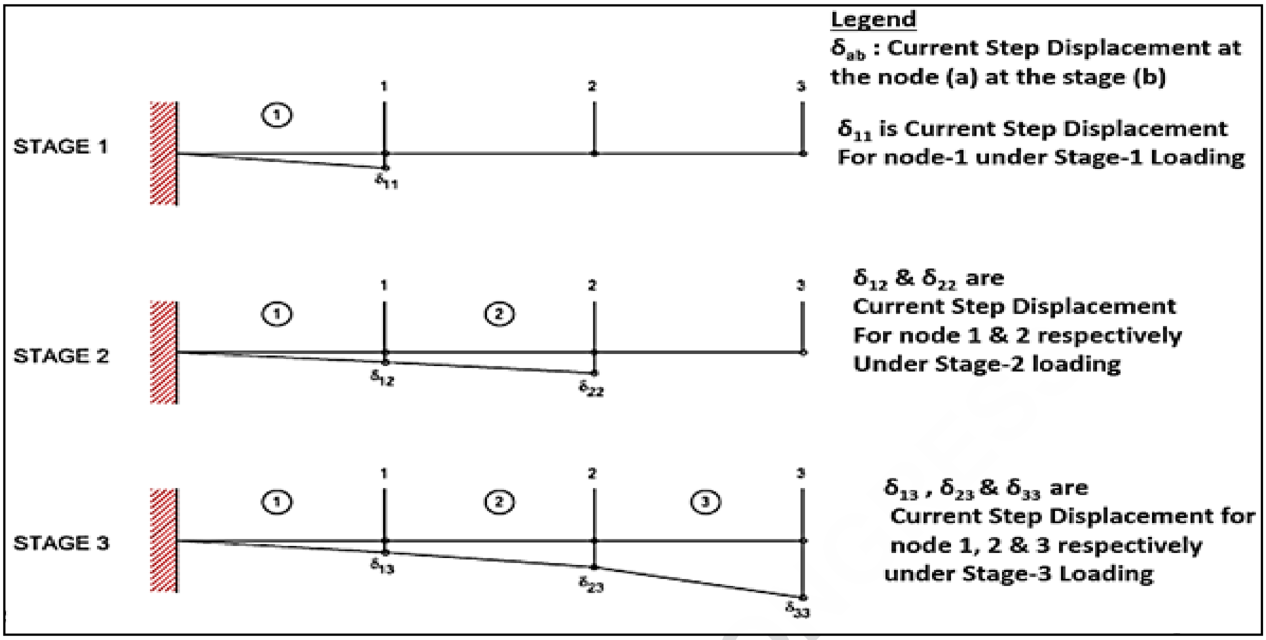


Fig 9.23 Net / Accumulated Displacement at Node (For Cast-in-Situ segments)

Net displacement profile at the end of erection of each construction step is shown in Fig-9.23 (top figure) if no precamber is done.

By reversing the final net displacement profile of erected unit at end of construction, one can work out precamber value which needs to be provided at each newly cast node in each construction stage w.r.t final proposed profile curve of segments as shown in Fig 9.24 (lower figure).

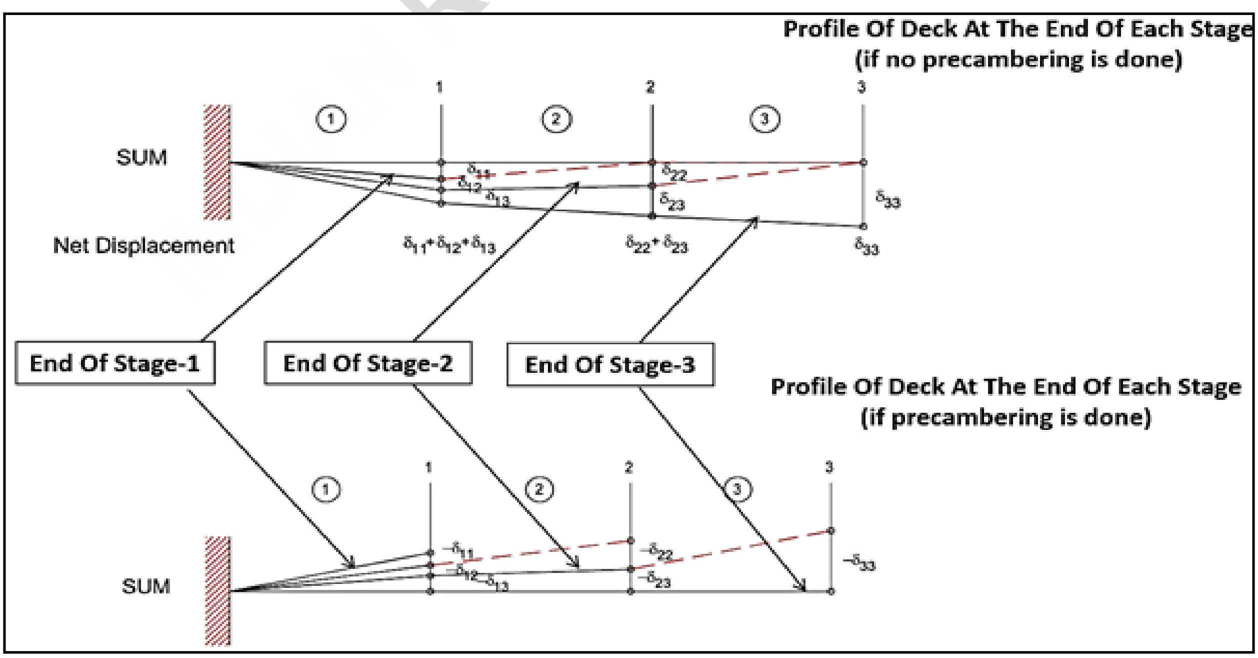


Fig 9.24 Precambering Using Net Displacement For Cast-In-Situ Segments

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1. Plot Adopted from Leonhardt & Zeller 1970
2. IRC:SP:114 – Guidelines For Seismic design Of Road Bridges
3. IRC:112 – Code of Practice For Concrete Road Bridges
4. IRC:6 – Loads & Load Combinations
5. IRC:24 – Code of practice For Road Bridges , Section V , Steel Road Bridges (Limit State method)
6. Stay cables Recommendations of French Inter ministerial Commission on Prestressing (Edition November 2001 in French, June 2002 in English)
7. EN 15129 Anti Seismic Devices
8. Eurocode EN-3 , Part 1-5: Plated Structural Elements

CHAPTER - 10

DESIGNING AND DETAILING ASPECTS

10.1 Introduction

In the preceding chapters, the different major components of the Extradosed bridges have been described along with their influence on the overall behaviour, effects and choices. In this chapter aspects of the design and detailing of the various interfaces between these components will be described in a general fashion with specific situations identified from the most common arrangements used in these types of bridges. In this chapter, the major focus is given to the design of the components with concrete as a material of construction for the deck. The major components discussed in this chapter relate to:

a) The connection between the pylon and the extradosed cables

b) The connection between the extradosed cable and the deck

c) The design of the interface between the deck and the pylon

d) The design of the connection between pier table segment and precast segment

e) The connection between inclined strut/tie in box girder at the web-soffit junction and at deck slab

In all these situations the aspects described here are simplified approaches in most cases and are correspondingly conservative in their approach. A more rigorous FEA is often a more accurate way to analyse and then design the sections.

10.2 The Connection between the Pylon and the Extradosed cables

This connection design is dependent upon the type of arrangement chosen for the cables. The primary fact with extradosed bridges is that the pylons are short, and the bridges are generally for span lengths between 80m and 250 m. In this span range, the arrangement is generally of the following types:

a. Cables running through the pylon with strands in individual tubes locked with friction

b. Cables running through the pylon with strands in individual tubes locked with the mechanical arrangement (Anti-slip)

c. Cables terminating on either side of the pylon with a connecting structural link

Fig. 10.1 to 10.3 show different type of connections between extradosed cable and pylon

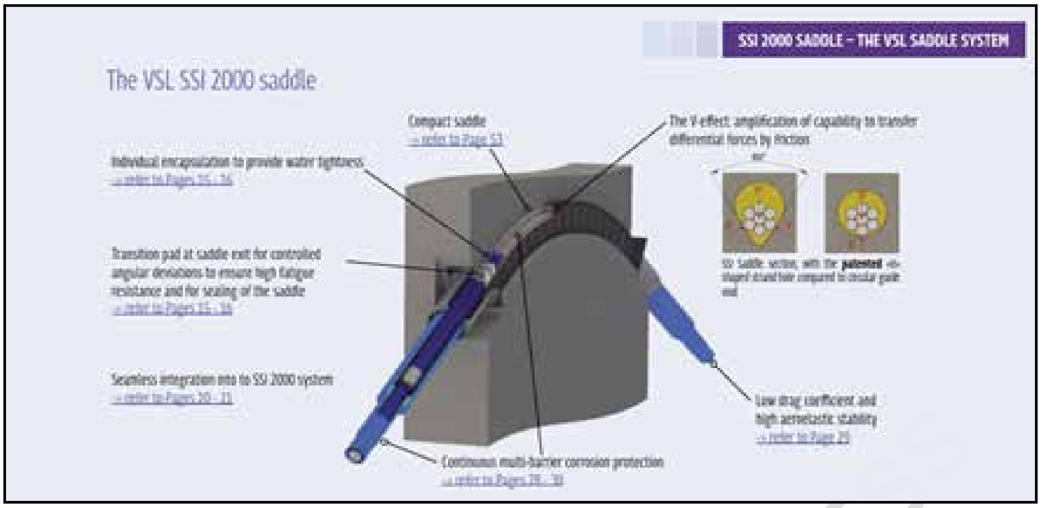


Fig – 10.1 – Cable Saddles enabled with Friction
(Image courtesy VSL Stay Cable systems 2018-12)

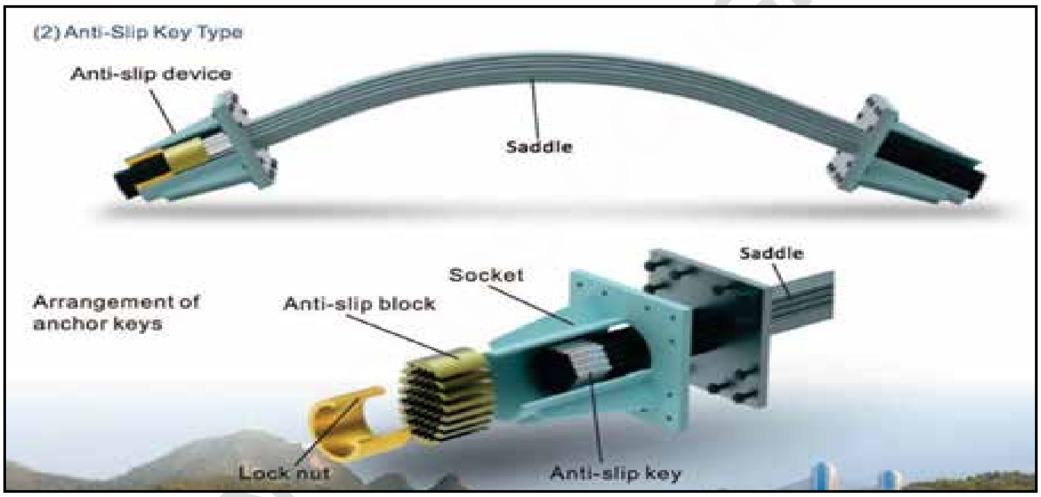


Fig – 10.2 – Cable Saddles enabled with Anti-slip block
(Image courtesy OVM Catalogue for OVM250 & OVMAT)

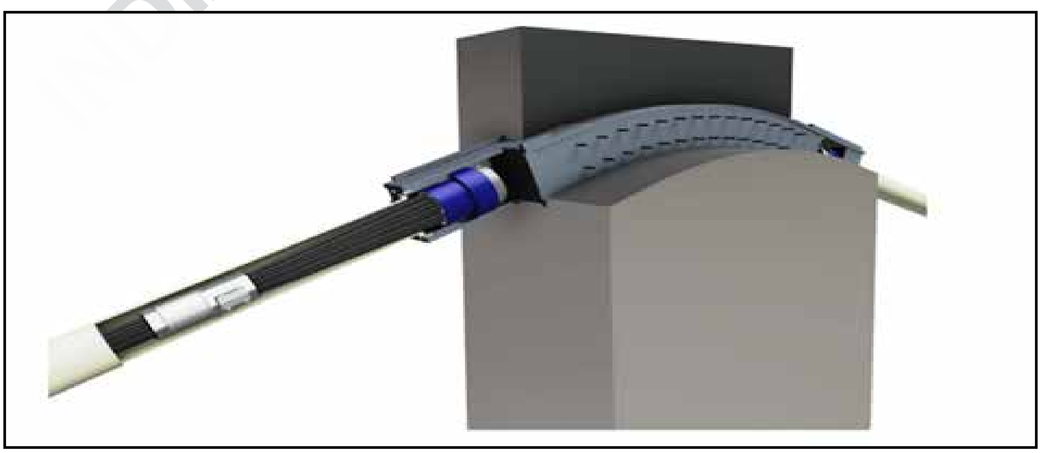


Fig – 10.3 – Cable with Connecting Structural Link
(Image courtesy DSI Catalogue)

10.2.1 Design Conditions

The Design conditions for these connections are in two levels i.e., their local load transfer and their overall lead transfer to the pylon. Obviously, the local transfer of loads leads to the overall forces in the Pylon. The local design conditions for this are:

a) Normal Service Condition

b) Cable breakage condition (Accidental Load)

In this section, the principles of the handling of these forces in the interface between the hardware and the pylon structure are delineated.

10.2.1.1 *Longitudinal forces*

a) The Normal service condition (Fig. 10.4) for these forces can be idealised as follows:

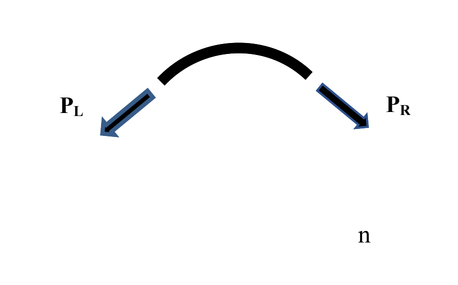


Fig. 10.4: Forces under Normal Service Condition

The difference in the force level in the left and the right-hand side cable could be due to either:

a) Imbalance in the permanent loads on either side of the pylon

Imbalance in the variable loads on either side of the pylon due to the various combination of variable loads

b) The Cable break condition (Accidental Load Case) (Fig. 10.5) can be idealised as follows:

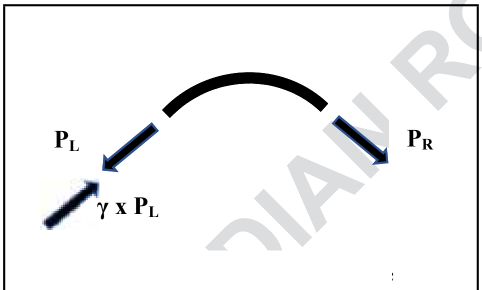


Fig. 10.5: Forces under Cable break Condition

The cable break situation causes effects differently for different hardware used in the pylon. In the case of the first two types of hardware viz. strands held in place by friction or an anti-slip device, the strand break leads to loss of tension on both sides, but in the case of the third type viz. cables terminating on either side each cable is treated as separate cable and the cable break situation is as shown in the figure to the left (Fig. 10.5) for the loss of Left-side cable.

In the figures 10.4 and 10.5 P_R and P_L are the forces in the extradosed cables on the right and left side of the pylon respectively when looking at the bridge from the sides.

The factor γ shall be 1.5 for the cable break situation to consider the dynamic effects. For cable replacement condition, γ shall be 1.0 (as it is a controlled operation).

The handling of the unbalanced forces within the hardware of the system is ensured by the system supplier, through either friction, an anti-slip device or the physical anchorage as described in the earlier section.

The unbalanced longitudinal forces can be in a simplified manner considered to be transferred to the adjacent concrete through the arrangement of shear studs. With a detailed analysis, the bearing of the part of the structural link to the adjacent concrete may be considered in this load transfer mechanism.

The Cable break condition is an accidental load condition, and the design of the studs shall therefore be carried out in the ultimate limit state as defined in the combinations and conditions of IRC 6 and corresponding provisions of IRC 24.

10.2.1.2 *Vertical loads (Fig. 10.6)*

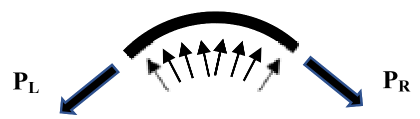


Fig. 10.6: Radial Pressure on Saddle / Link

Fig. 10.6: Radial Pressure on Saddle / Link

The vertical forces due to cable inclination are resisted by the reaction from the surrounding concrete. In the case of steel pylons, the load transfer mechanism shall be as per the details of the connection between the hardware and the pylon.

In the case of concrete pylons, the radial pressure is transferred to the concrete through the bearing of the hardware on the concrete. This situation is handled through the design provision for allowable bearing force as per Cl.16.11 of IRC 112 and N_{Ed} shall be less than F_{Rdu}. N_{Ed} is the vertical component of the cable forces and the radial forces transferred from the upper saddles on top of saddle under consideration, and F_{Rdu}, is the capacity of the loaded area underneath saddle to sustain the concentrated force N_{Ed}

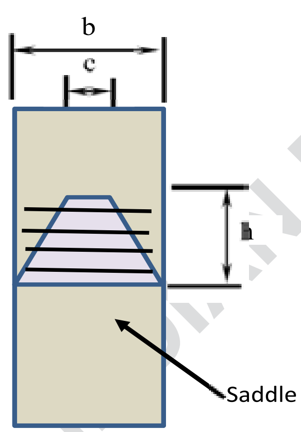


Fig. 10.7: Radial Pressure on Saddle / Link

The concentrated forces due to the vertical component causes transverse splitting action and this splitting force is evaluated as below (the expression is taken from Cl. 16.15.1.4 of IRC 112).

The splitting force F_s may be calculated as follows:

$$F_s = 0.25\,(1 - c/h)\,N_{Ed}$$

In case **h** satisfies the requirements in CL. 16.11 given in the Fig. 16.9 of IRC 112 related to concentrated load transfer in the direction of dimension **c**, the splitting tensile stresses may be deemed to be taken care of by the tensile strength of concrete itself. If **h** does not satisfy the requirements of CL. 16.11 of IRC 112 mentioned above, then tensile reinforcement distributed over the height **h** shall be detailed to take care of the splitting force F_s. When required, the minimum diameter of reinforcement to take care of the splitting force shall be 12 mm.

10.3 The Connection between the Extradosed cables and the Deck

The connection between the Extradosed cables and the deck needs to be designed primarily to ensure the transfer of the cable's forces to the longitudinal load transferring elements such as an edge beam or the webs of a box section etc. This connection can be achieved either by a direct connection to such an element or an indirect connection. In most decks, it tends to be indirect because of the constraints on the deck geometry, functional requirement of the deck area and the width of the deck. The cables are either placed in the central plane if the functional requirements of the deck permit it or on the sides if so required.

10.3.1 *With Central plane of extradosed cables*

Typically, these are found on roads with divided carriageways, which enable the provision of space for the central plane of cables, within the central median. The provision of vehicular barriers on either side of the central median also acts as vehicular impact protection to these cables. The connection design depends upon the arrangement of the extradosed cables. The cable arrangements can be either in a single plane at the centre of the cross-section or in two planes on either side of a central web. Sometimes even in the absence of a central web, a pair of cables is also adopted to reduce the size of cables to be used. In either case, the design of the connection aims to transfer of the concentrated cable forces to the webs. In these conditions generally, a full depth or a part depth diaphragm is provided for the transfer of the concentrated cable forces to the set of webs. The part depth diaphragm is generally proportioned to enable the movement of persons from one side of the diaphragm to the other. This also facilitates the shifting of the stressing jacks within the section from location to location.

In some cases, it is also possible that there are no central webs and the extradosed cables transfer their forces to webs through only the deck slab or through a set of ties either in steel or concrete (reinforce or prestressed) to the webs.

10.3.1.1 *Cable Force transfer through diaphragms*

The central plane of cables with diaphragms can also have a variety of design conditions depending upon the width of the deck and the cross section.

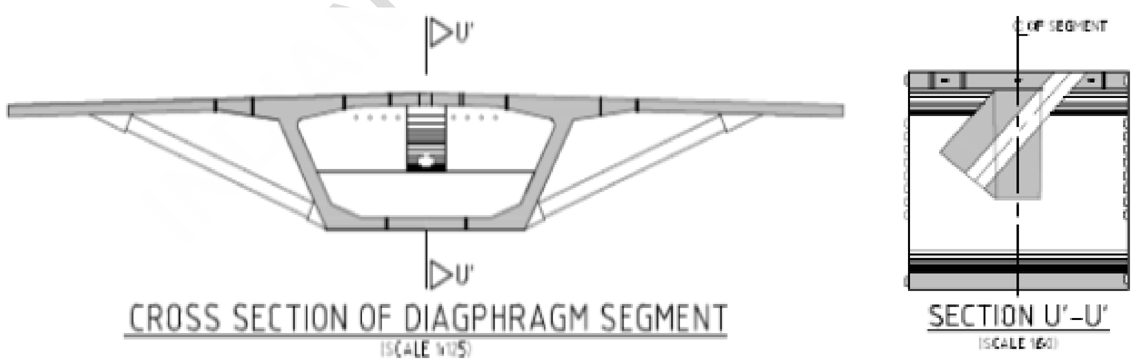


Fig. 10.8 - Typical Connection between central cable and narrow mono cell deck

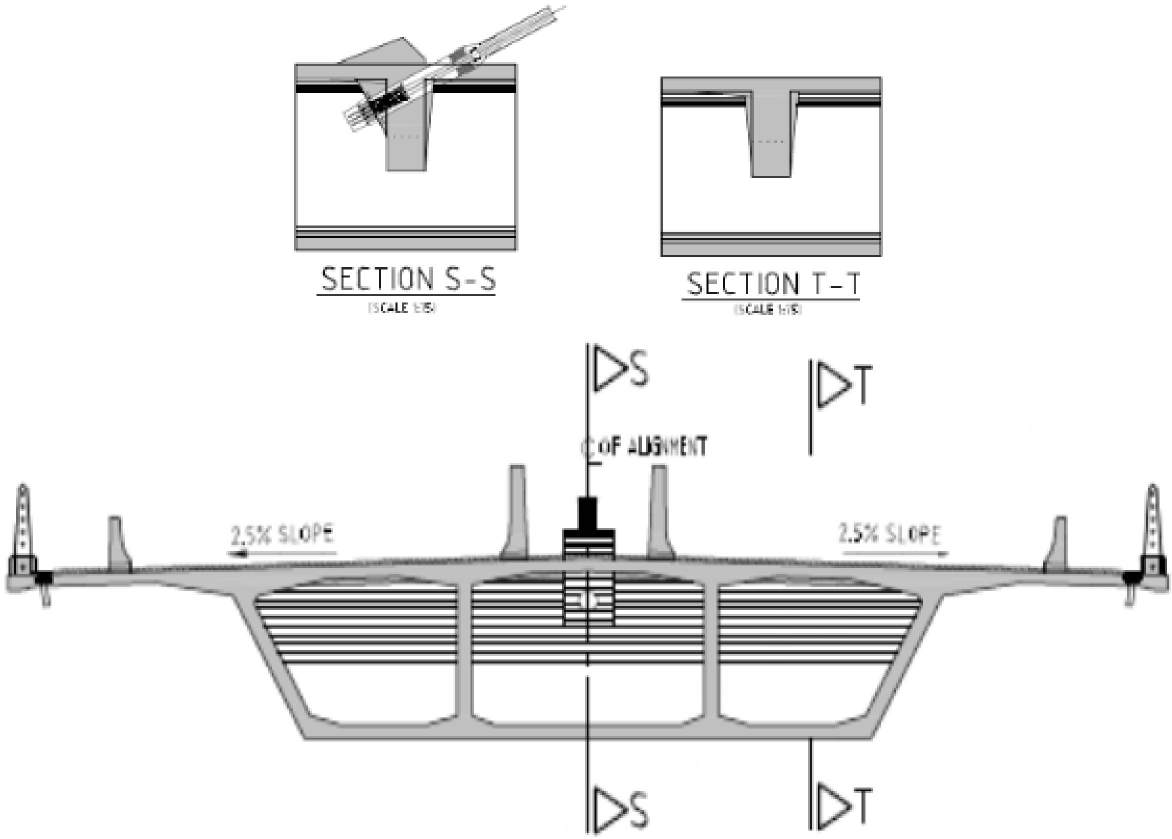


Fig. 10.9 - Typical Connection between central cable and wide multi cell deck

10.3.1.2 *Design Idealization in-plane forces*

For the above cases the design idealisation can either be done through a detailed finite element idealisation or through a reasonable simple and conservative local design model.

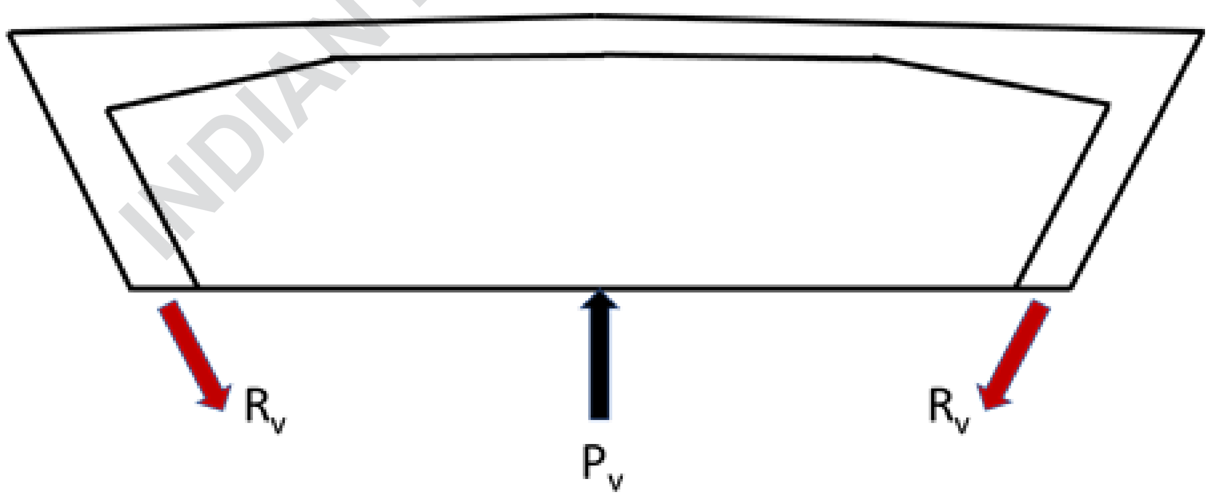


Fig. 10.10 – Simple Idealization for the design of the Diaphragm for Central cable and narrow Mono cell deck for in plane forces

In Fig. 10.10, P_v is the vertical component of the extradosed cable force, R_v are the reaction in the webs.

The design process for this diaphragm element for the in-plane forces can be carried out by either bending theory or strut and tie method depending upon the geometry. Detailing of the main tension reinforcement should consider the geometry and possible deep beam behaviour.

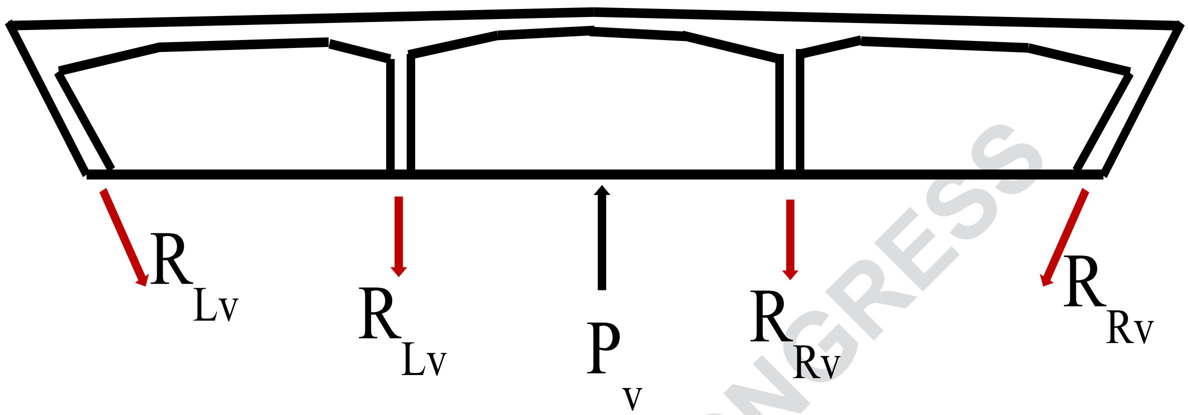


Fig. 10.11 – Simple Idealization for the design of the Diaphragm for Central cable and wide Multi cell deck for in plane forces

In Fig. 10.11, P_v is the vertical component of the extradosed cable force and R_{Lv} and R_{Rv} are the reaction forces for this vertical force in the left side and right-side webs

The design process for the in-plane forces of this diaphragm element can be carried out by bending theory. Detailing of the main tension reinforcement should consider the geometry and possible problems with concreting. Prestressing the Diaphragm in transverse direction is also possible to reduce reinforcement congestion.

For partial height diaphragms the reaction forces R will require suspension reinforcement distributed over the effective length of section along the box that transfers the whole of Pv. The effective length of box to distribute the suspension reinforcement may be determined by simpler effective width concepts adopted for the loads transmitted to the slab through the diaphragms. This approach might also be partly consistent with the transverse bending effects caused by the concentrated cable forces.

10.3.1.3 *Design Idealization out of plane forces*

This design condition can either be done through a detailed finite element idealisation or through a reasonably simple but conservative local design model.

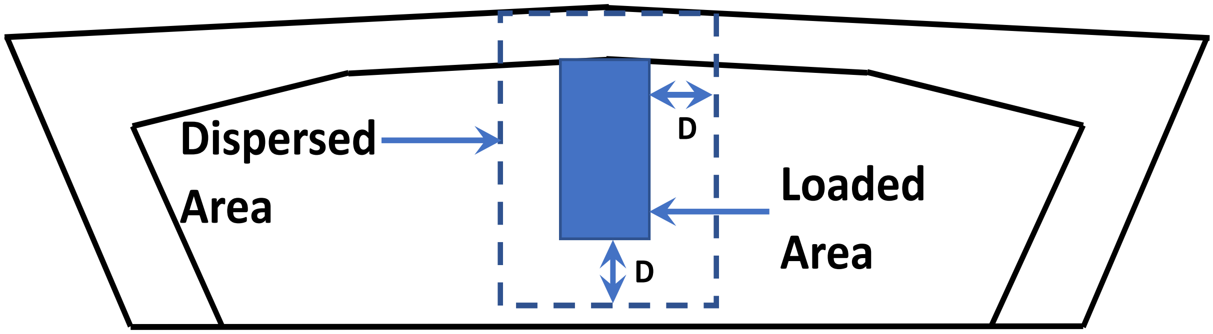


Fig. 10.12 – Simple Idealization for the design of the Diaphragm for Central cable and narrow Mono cell deck for out of plane forces

The diaphragm shall be checked for punching shear based on the loaded area geometry as per the provisions of Cl. 10.4 of IRC 112. The bending effects on the plate can be evaluated by the patch loading on the dispersed area which is equal to the symmetrical area formed by a perimeter at a distance D away from the loaded area. D being the thickness of the diaphragm.

Similar treatment may be adopted for the multicell decks also.

10.3.1.4 *Cable Force transfer through ties*

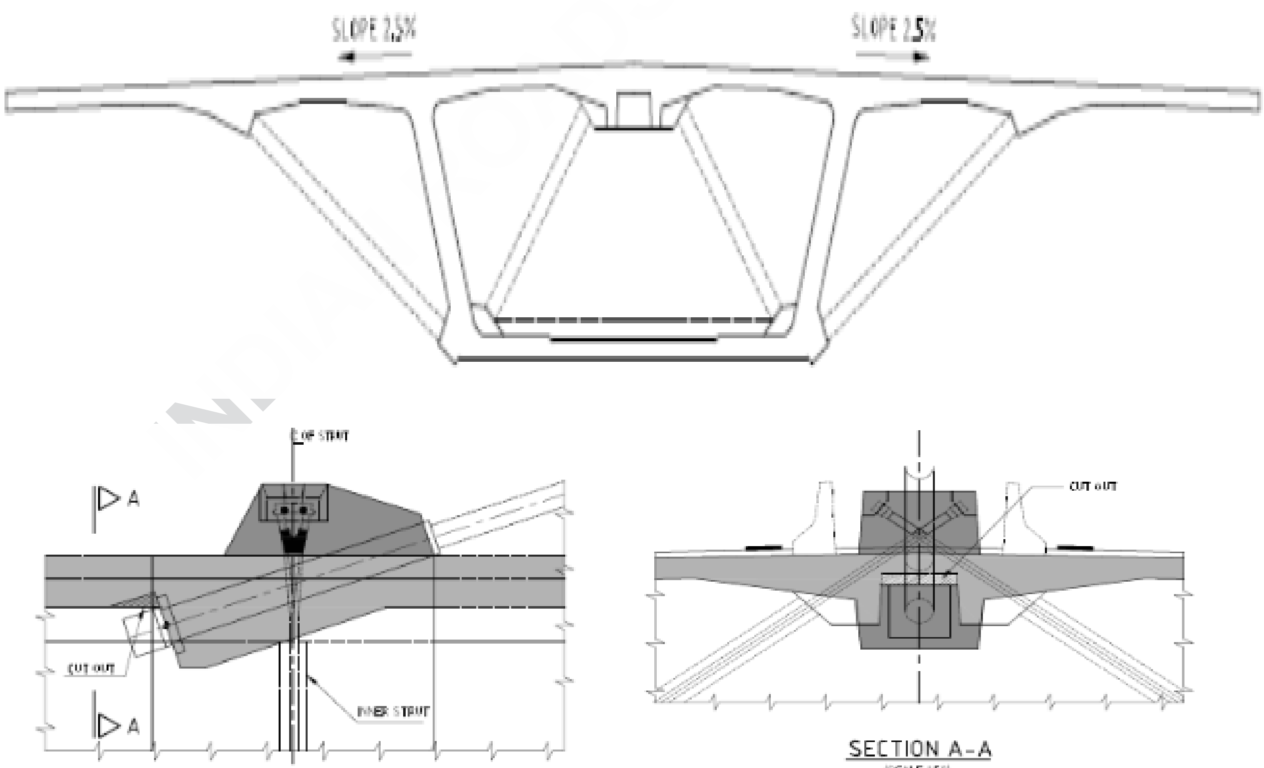


Fig. 10.13 - Typical Connection between central cable and wide mono cell deck

10.3.1.5 *Design Idealization of in plane forces*

In this situation the ties can be designed either in steel or in concrete (reinforced or prestressed). In extradosed bridges with shallow angles of cables the vertical component is not very large and hence the design process for the tie elements by a simple truss analogy is quite often sufficient as shown in Fig. 10.14. In case the tie member is prestressed to transfer the cable tension directly to the web bottom, the dimensioning of the junction between the webs and soffit should be done carefully to avoid clashes with other longitudinal cables that may be running along there. It must be noted that these struts are also subject to compression under the action of self-weight of the section, superimposed dead loads and live loads. The tensile forces in the cables are caused due to stressing as well as due to live loads and hence this simple approach might be conservative, and a better estimate might be obtained through a sectional FEA modelling over limited area.

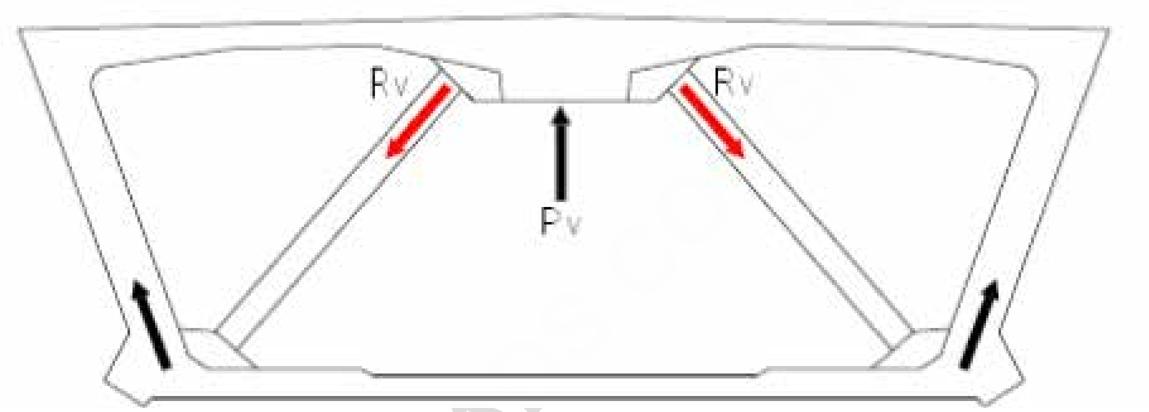


Fig. 10.14 – Simple Idealization for the design of the Tie for a Central cable and wide Mono cell deck

The force P_v is the vertical component of the cable force and R_v are the equilibrating tie forces transferring the vertical forces to the web bottom (Fig. 10.14).

10.3.2 *With Two planes of cables on sides*

The side planes of cables too can also have a variety of design conditions depending upon the width of the deck and the cross section. One such arrangement for a 25 m wide deck is shown in the fig. 10.15

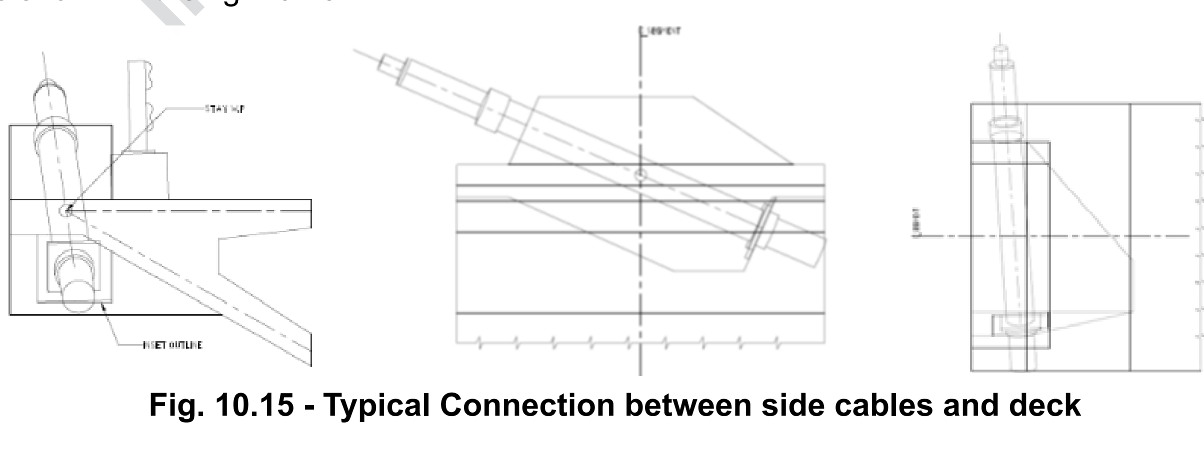


Fig. 10.15 - Typical Connection between side cables and deck

Apart from the above there are several other possible arrangements. It is not possible to cover all the possibilities, but it some of the other arrangements that can be seen in different projects all around the world are as shown below.

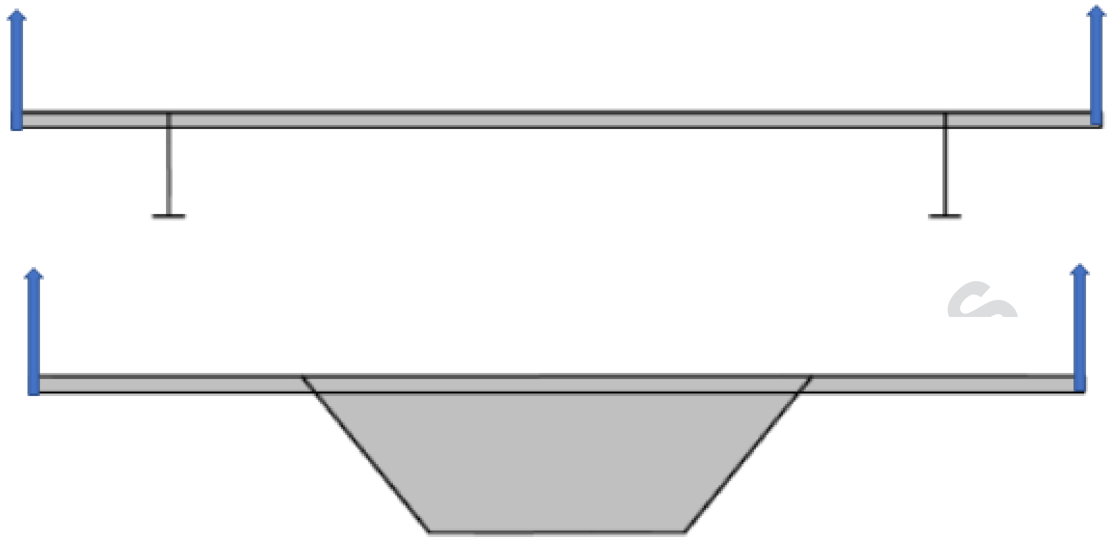
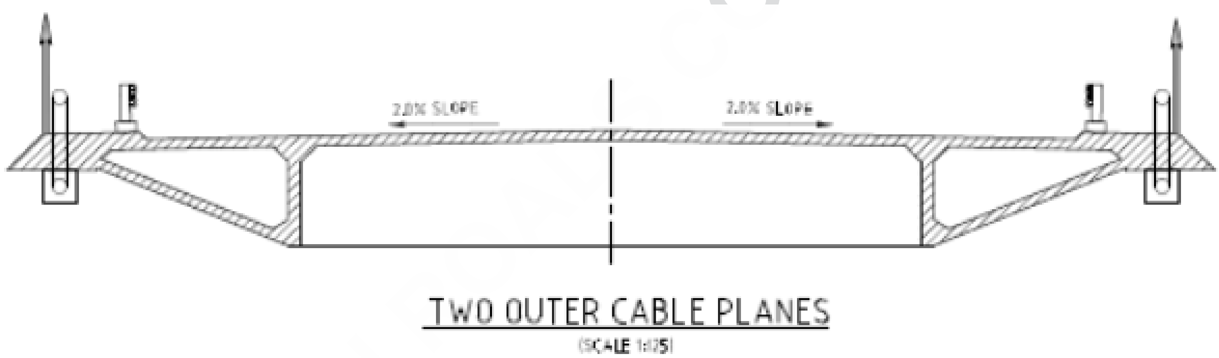


Fig. 10.16 - Typical Connection between side cables and deck through slabs



10.17 Typical Connection between Side deck cables and Edge beams with Open concrete cross-section

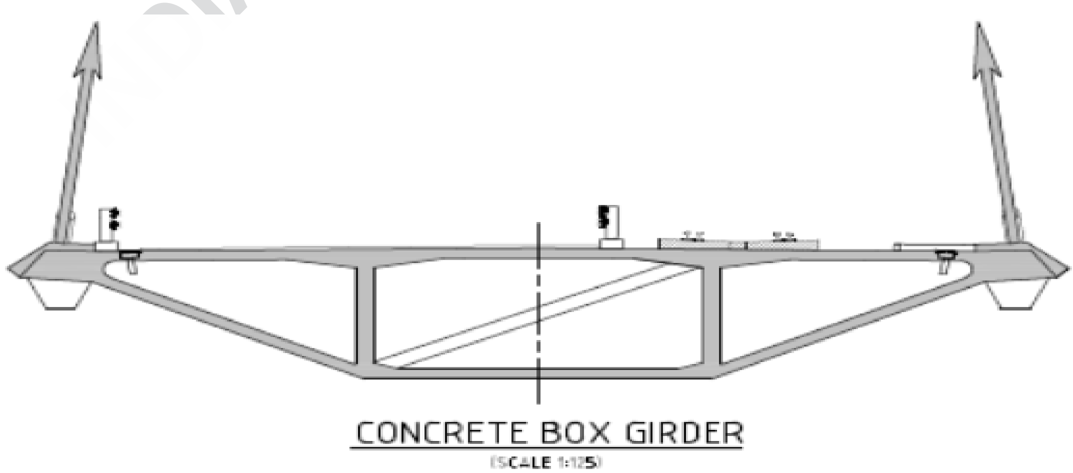


Figure 10.18 Typical Connection between Side deck cables and Box girder section

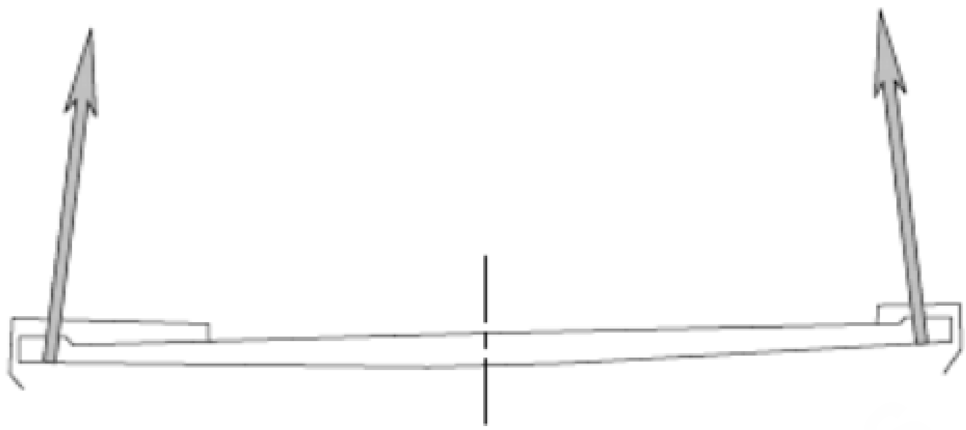


Figure 10.19 Typical Connection between Side deck cables and solid slab section

10.3.2.1 *Check for Interface shear between the cables and the deck*

In all these arrangements there is a common element that is the cable housing which accommodates the replaceable cable anchorages. The dimensioning of this housing component depends upon the cable inclination, length of the segment in which it is being installed (in case of segmental construction) and the size of the element of the deck to which it is being connected.

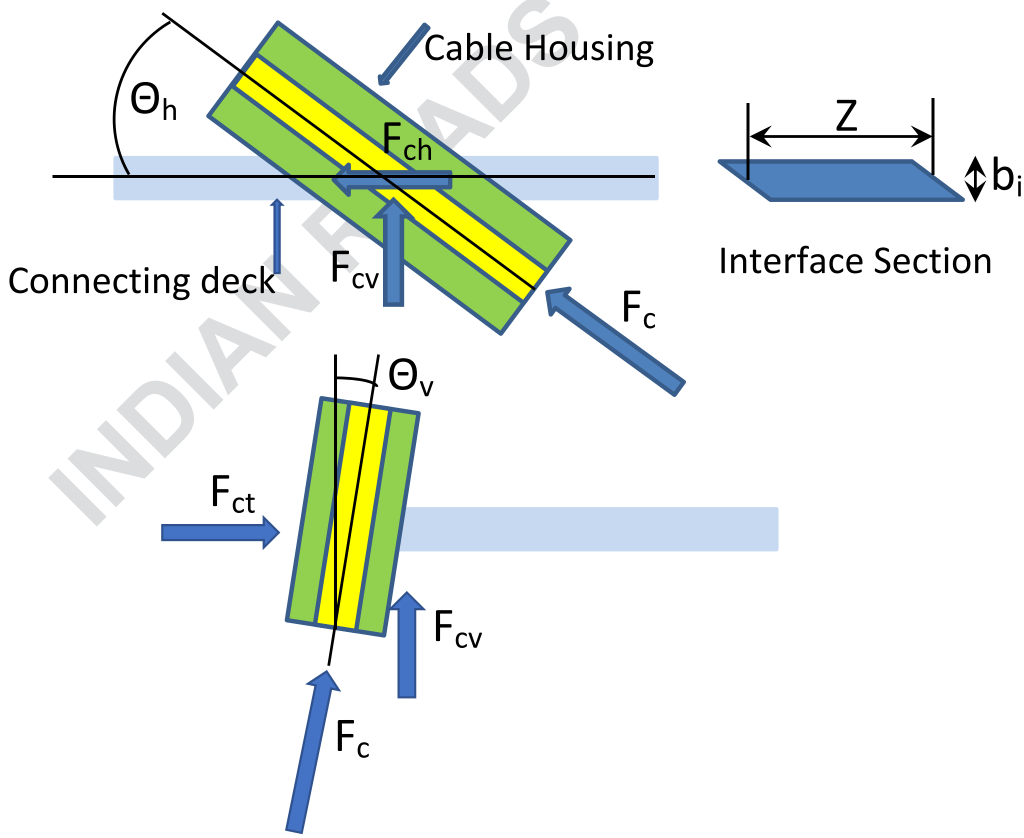


Figure 10.20 Interface shear check between cable housing and Deck

In Fig. 10.20, F_c is the cable force, θ_v and θ_h are the inclination of the cable with the horizontal and the vertical plane respectively, F_{cv} and F_{ch} are the components of the cable forces in the vertical plane and the horizontal plane respectively. Z is the interface width of the cable housing with the deck and b_i is the thickness of the deck at the interface.

The first safety check that needs to be performed is the interface shear capacity check between the cable housing and the connecting deck in accordance with the Cl. 10.3.4 of IRC 112. It may be noted that the referred clause deals with concrete placed at different times, however, the same may be adopted with a μ of 0.9 in case the cable housing it is cast monolithically with the deck. In other situation guidance may be taken from the referred clause. The effect of the vertical deviation of the cables from the deck should be duly accounted for in the reinforcement determination at the interface. The impact of μ may not be very significant as it relates to the minimum co-existing σ_n which is depending upon the magnitude of F_{ct} (caused due to inclination of the cables with the vertical plane θ_v) which is generally very small.

The value of F_c should in accidental condition also cater for the sudden loss of the cable anchored there with suitable γ defined in section *10.2.1.1*.

10.3.2.2 *Check for Shear in the deck slab*

The force F_c described above will then get transmitted to the nearest web through any of the following actions:

a) Shear through the deck slab

b) Shear through the deck and stiffening diaphragm

c) Shear through the deck and tie beam/slab

In case of the transfer of shear to the deck slab the shear may be assumed to be taken up by the portion of the deck at the face of the nearest edge over the distance as derived form the details given in Fig. 10.21.

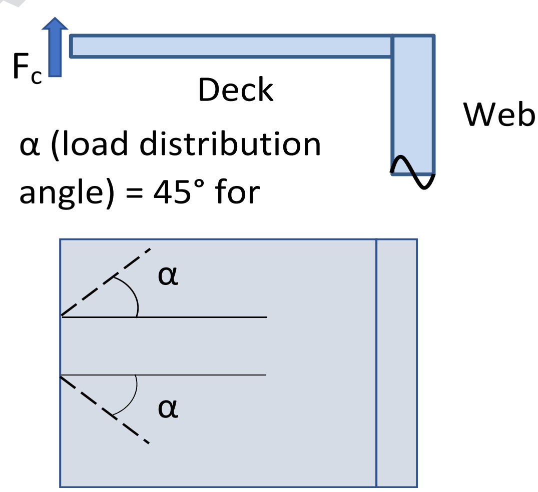


Figure 10.21 Load distribution through the Deck

The above simplified rule is extrapolated from the recommendation of Cl. 7.3.3.1 of MC 2010.

The capacity of the deck slab at the effective section from the face of the web may be verified in the conventional manner as per IRC 112

10.3.2.3 *Design for transfer the forces to the webs*

Depending upon the transfer of this cable force to the deck slab, i.e., at the top of the web or at the bottom of the web, additional design rules will apply. In case the cable forces are transferred near the top of the web, then suspension reinforcement in addition to the shear reinforcement in the web would be required within the distribution length defined above.

This transfer of the vertical component of the cable force from the deck to the nearest web may be approximated by the rule in *10.3.2.2* for transfer through slabs/slab cum stiffening beam. In case of presence of an inclined soffit or external strut/tie beam, this force could also be transmitted to the webs through a set of prestressing forces similar to the concept defined in Cl. 10.3.1.5 for internal ties.

10.4 Design of the interface between the deck and the pylon

This interface is through the element sometimes called Pier table. The pier tables are those portions of the deck that are the parts of superstructure to be cast first. Due to the small vertical stiffness of the extradosed cables, offering little advantage in the control of the profile during construction, the pier tables are monolithic / integral with the pylon to get rotational stiffness for profile control. There are some bridges though very few in number where the deck is supported on bearings fixed to the cross member in the pylon to transfer the forces from the superstructure to the lower pylon. Otherwise, in most cases, the transfer of forces from the deck to the lower pylon is through the integral connection, which is referred to here as the pier table.

The design of this portion of the deck is special because it does not fit into the general bending sections. Also, this portion of the deck is quite large to be treated as a simple joint that is treated in certain sections of the Eurocode or other design code. It is obvious that this pier table's local design is dependent upon the deck geometry and the type of pylon arrangement. In the following section, a few of the typical pier tables are presented for design and detailing consideration

10.4.1 *Single plane of pylon*

It is evident that this part of the deck will be designed for all the forces that are to be transferred to the lower pylon from the superstructure. To summarize the forces that are transferred to the lower pylons through the pier table are

1. Shear through the webs
2. Torsion through the webs, deck and soffit slab
3. Self-weight of and other local loads on the pier table itself.
4. Unbalanced longitudinal moments

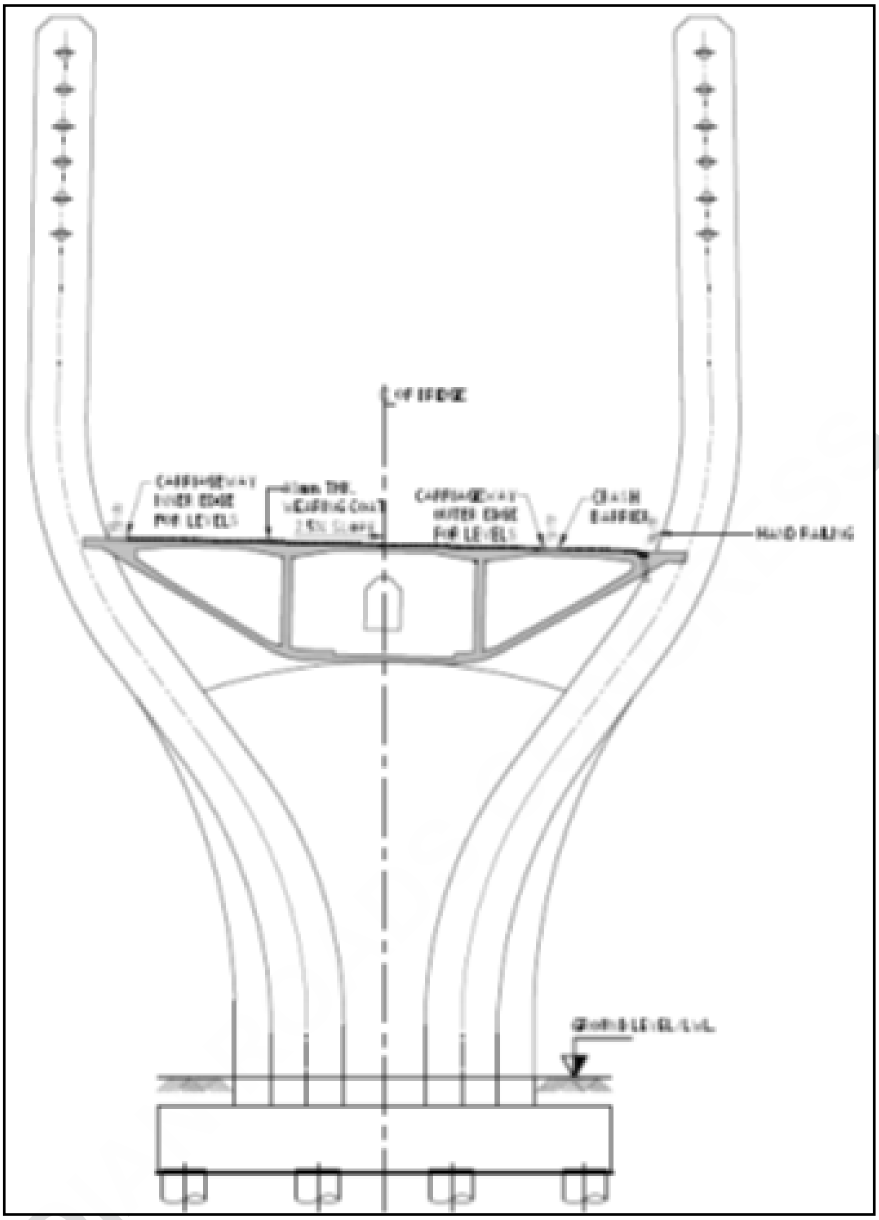


Figure 10.22 Typical single leaf lower pylon with cable in two planes

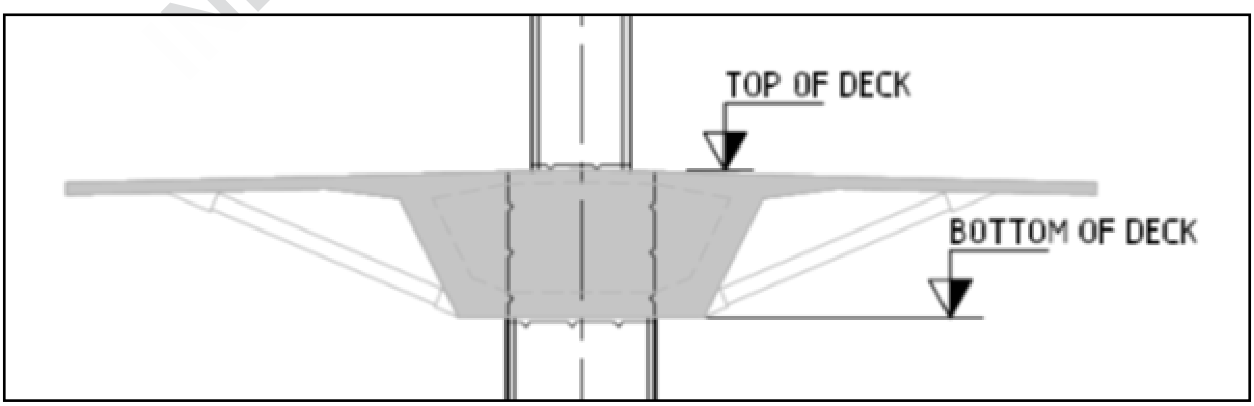


Figure 10.23 Typical single leaf lower pylon with cable in central planes

It is necessary to carry out local designs generally through local strut and tie model depending upon the size of the basic pier table section or a more conventional frame analysis in case the deck width is large, and the pylon is arranged with twin planes of cables.

10.4.2 Twin leaf pier

A twin leaf lower pylon is also a common arrangement and its junction with the deck particularly a box type deck leads to a special design condition for the transfer of the unbalanced longitudinal moments to the twin piers through the connecting

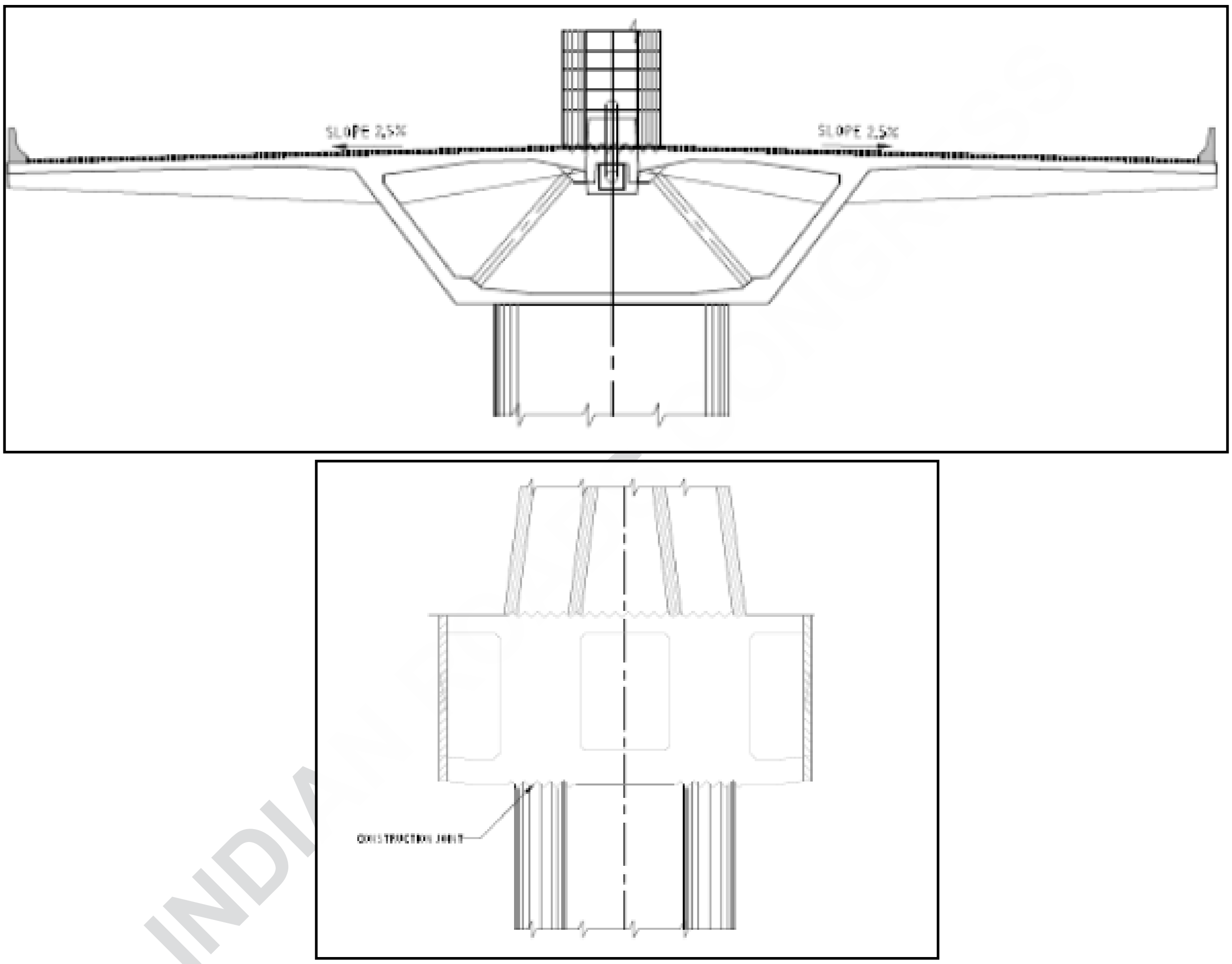


Figure 10.24 Typical twin leaf lower pylon with cable in central plane

In the longitudinal direction the walls between the two leaves of the pylon is instrumental in transferring all the unbalanced shears and moments to the twin leaves of the lower pylon. This aspect requires a local Strut and tie design considering the direction of the imbalance in the shears on the left and right side of the lower pylon. A qualitative idealization is shown in the figure 10.25.

The Blue walls between the soffit and deck slabs are designed to transfer the torsional forces in the cross section to the lower pylon (yellow) below the soffit slab. The torsional forces are represented by the shear flow in the web walls. In all the local strut and tie model for the transfer of shear forces to the twin leaf lower pylon there is a necessity for suspension reinforcement at the junction of the webs, side walls (green) and the diaphragm walls (blue). Generally, this does not require additional reinforcements if the flexural reinforcement present in the lower pylon, and the shear reinforcement in the span side of the webs and some minimal vertical reinforcements in the diaphragm walls are extended to the top of the deck.

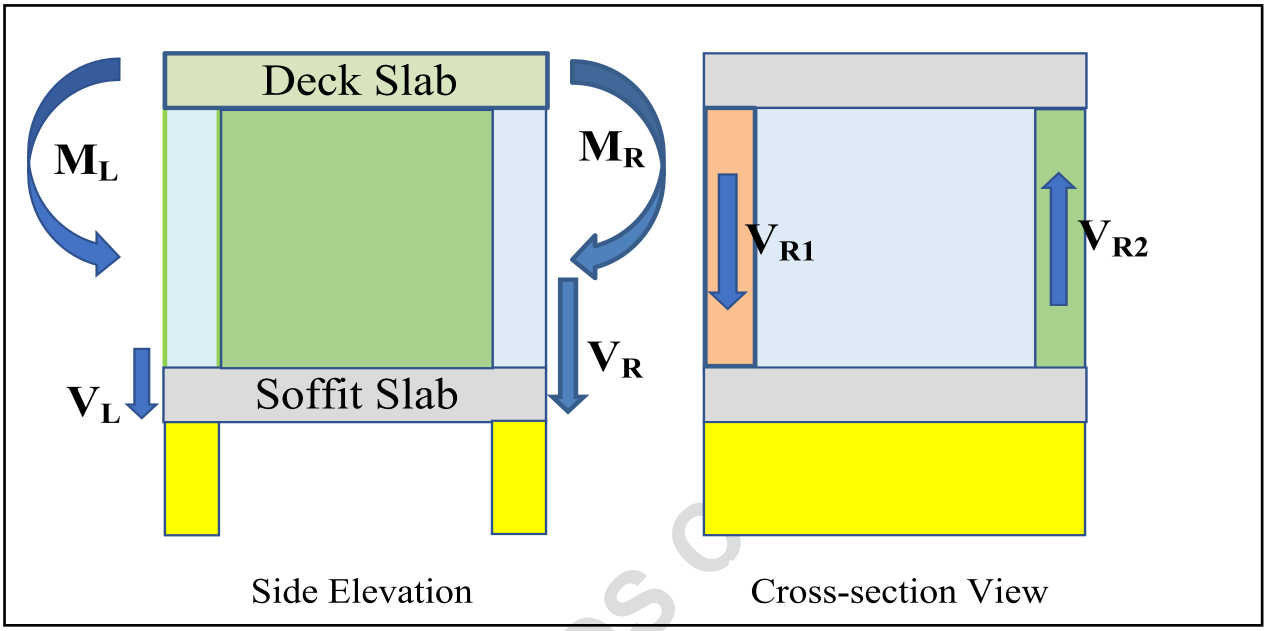


Figure 10.25 Load idealization at the junction of twin leaf pylon- and Superstructure

The design of the webs between the twin diaphragms (blue walls) may be carried out by using Strut and Tie model (Fig. 10.26)

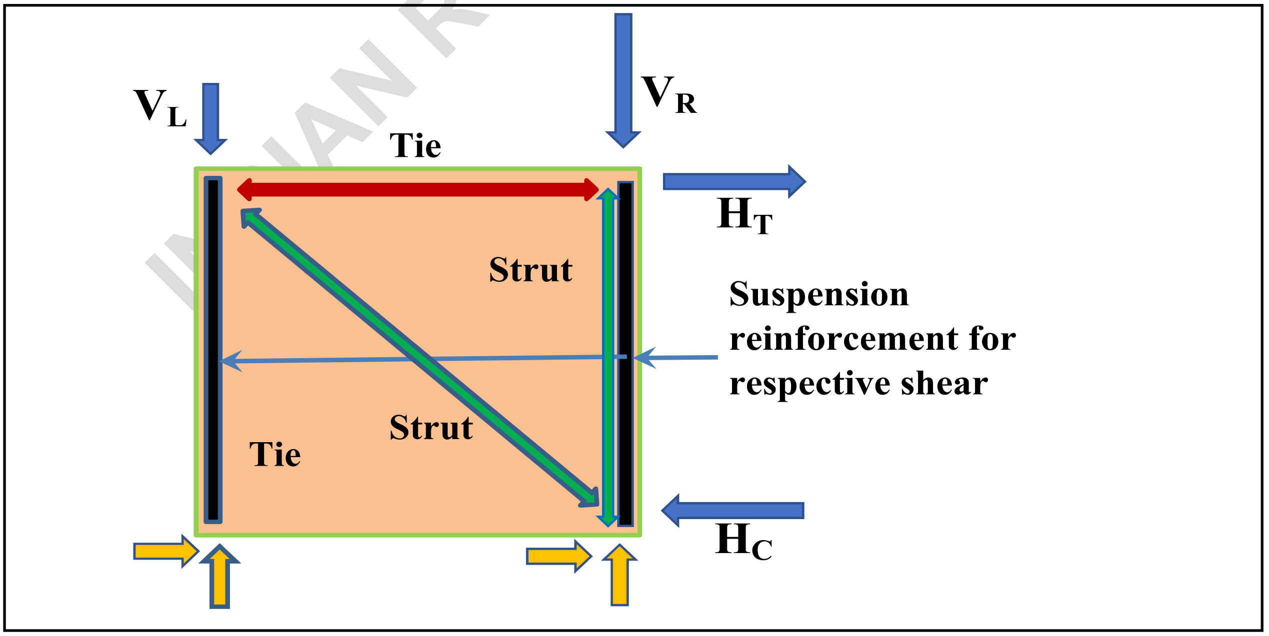


Figure 10.26 Strut & Tie Model for Design of Web wall between the twin leaf piers

The suspension reinforcement shall be distributed over the prescribed width define in Cl. 16.13 (2) of IRC 112. The tie in the above model will generally not require any additional strengthening as the main tension strength in the deck slab will be present between the two diaphragms also and the fact that the differential shear is primarily a manifestation of differential bending between the two sides of the pylon. H_T and H_C are the tension and compression force couple due to the imbalance in the longitudinal moments on the left and right side of the pier table.

10.5 Design of connection between pier table segment and precast segment

In case the extradosed bridge is planned to be built with precast segments and as described in previous section the pier table happens to be monolithic with the pylon and hence cast in-situ, integration of the adjacent precast segment with the in-situ pier table is invariably carried out with the provision of an in-situ stitch between the pier table and the precast segment. It is essential that the segment handling system have necessary mechanism to position the precast segments accurately both in elevation (level) and orientation (in vertical plane as well as horizontal plane). The absence of such arrangement will lead to misalignment in the profile, which may necessitate correction using shims and other devices to achieve the desired alignment (vertically and horizontally), when detected.

The in-situ stitches are generally unreinforced with the gap between the pier table and the precast segment between 100 to 250 mm.

CHAPTER - 11

INSPECTION, MAINTENANCE, AND STRUCTURAL HEALTH MONITORING

11.1 Introduction

Performance of extradosed bridges during design service life shall be achieved by implementing a maintenance management plan drafted during the design stage. Though the principles of maintenance of extradosed bridges are not different from girder bridges, inspection and maintenance of components such as extradosed cable systems, deck, and pylon are very important to ensure the level of service during the design life.

The Main Tension Elements (MTE) within a cable bundle are, in most cases, hidden from the view of inspectors. Access to cables for visual inspections or non-destructive testing (NDT) is generally difficult, and in the case of the anchorage zones, nearly impossible. Those who are tasked with the inspection and maintenance of stay cables are faced with challenges for which proven and accepted methodologies and tools are limited and, in many cases, very costly.

This Chapter focuses on deck, extradosed cable, and pylon inspection, maintenance, and SHM of extradosed bridges. This includes the overall framework of SHM, the design of SHM system the parameters that can be monitored, and the corresponding sensors, data management, and interpretation of results. SHM may be used to verify the design assumptions, to evaluate the performance of the bridge during its service, especially under extreme events, and when exposed to various environmental conditions. This could be used for the maintenance management of bridges. For inspection and maintenance of substructure, foundation, and other auxiliary components, IRC: SP: 35 and IRC: SP:18 may be referred.

11.2 Inspection

Regardless of the quality of design and construction, it is inevitable that a bridge structure will suffer some degradation due to the effects of age, environment, and loading that often exceed the design requirements. The extent of this degradation is also influenced by the extent of the inspection and maintenance programme. The design life of long-span bridges is generally achieved by providing durable components, detailing for easy inspection access, having regular inspection and maintenance performed, and replacing components that have reached their service lives. Therefore, inspection access shall be an integral part of the design. Adequate space should be provided to allow access for construction, inspection, and maintenance personnel. The space should also allow for material and equipment to be delivered from the deck or ground level to the cable anchors for repair and replacement work. Generally, appropriately sized access openings and interior lighting or a power source should be provided. The design should provide sufficient points of access and minimum opening sizes for inspection and maintenance.

Some of the extradosed bridges have many similar characteristics to the externally prestressed girder bridge and as such requires a similar maintenance regime. The major difference between these systems is that the extradosed girder has an effective continuation of the external prestress cables extending above the deck level and anchored to the cable pylons. This arrangement, therefore requires a different access system during inspection and maintenance periods. However, as the pylons are shorter than cable-stayed bridge pylons, they will be accessible by means of standard access equipment.

The organisational setup and training requirements of personnel for inspection and maintenance are given in **Annexure- A.7.**

11.2.1 *Scheduling of Inspections*

Various type of inspections and frequency as given **Table 11.1** may be used if not incorporated in the operation and maintenance manual.

Table 11.1 Scheduling of Inspection

Types of Inspection	Inspection goals	Frequency
Initial inspections	The first inspection is to be performed and inventorized before the bridge is opened to traffic to identify whether there is a change or update in the inspection procedure for future maintenance.	One time
Routine Inspections	Visual inspection of the components given in Table 11.2.	Initially, the periodicity could be once a month for six months. Subsequently, the duration between two inspections could be increased to 6 months, depending on the condition of the bridge component.
Detailed inspections	This should be a visual inspection from a close distance, NDTE could be used to identify the cause of distress (see Table 11.2)	Once in three years/ or when major distress is observed during the routine inspection.
Special Inspection/ Emergency Inspections	Inspections carried out immediately after the earthquake, cyclone, flood, fire, impact damage, etc. Can be done by foot with the naked eye or with binoculars and with the help of NDTE (see Table 11.2)	After natural /man-made disasters such as earthquakes, floods, cyclones, fire, etc.

Table11.2: Type of Inspections and parameters to be checked

Bridge component	What to inspect/ measure	Inspection type				
		Regular		Special/ Extraordinary After		
		Routine	Detailed	EQ	cyclone	other
Deck	Deflection/level	▲	√	▲	▲	▲
	crack	√	√	√	√	√
	vibration	▲	√	▲	▲	▲
Pylon	crack	√	√	√	√	√
	tilt/inclination	▲	√	√	▲	√
Extradosed cables	vibration	▲	√	▲	√	▲
	tension	▲		▲	▲	▲
Cable pipe	Damage Deformation Discolouration	√	√	√	√	▲
Cable Anchorage	Damage, deformation, corrosion	√	√	√	√	√
Vibration control devices	Damage Deformation Rust or deterioration Leakage of viscous material	√	√	√	√	▲

√ Perform the inspection, ▲ Perform inspection if needed

11.2.2 *Inspection Program*

A typical inspection program of extradosed bridges would include but not be limited to the following:

(i) Change to structural geometry

(ii) Deflections due to dead load and live loads.

(iii) Inspection for unusual cracks or deformations to concrete particularly in high-stress areas

(iv) Distress/damage of expansion joints and uneven road level on either side of the joint

(v) Evidence of corrosion to reinforcement, other embedded steel and prestress / extradosed cable

(vi) Erosion and abrasion of the substructure due to fast-moving particles / objects in the river

(vii) Collision damage

(viii) Fire damage

(ix) Deterioration of concrete due to chemical agents

(x) Integrity of corrosion protection

(xi) Scouring of foundation

The design reports along with “As Built Drawings” and construction records form the basic data for maintenance during the operation stage and it is necessary to store data for a long period.

It is very much necessary to measure and record the initial inclination of the pylon and cable tension during the initial inspection of the bridge, as they form the basis to identify the abnormality of these components. It would be appropriate to perform the maintenance rationally and efficiently by creating a database of various records, to make it possible, to refer to updated records of repair, inspection, and monitoring at any time.

For the cable system of extradosed bridge, for recording the finding during inspection the checklist given in **Annexure -A.8** shall be used.

11.3 Maintenance

11.3.1 *Maintenance issues to be considered during the design*

Maintenance issues that need to be addressed during the design stage are as follows-

i) Identify the design details that give rise to structural and durability problems, ii). Design details that cause difficulties for inspection, iii). Provisions of access for inspection, and iv) Inspection traveller(s) /underslung traveller/ maintenance carrier if required in case mobile bridge inspection cannot be used for bridge inspection.

11.3.2 *Maintenance Manual*

Contract documents shall specify the need for a maintenance manual. A maintenance manual should be seen as an integral part of the design, as regular inspection and maintenance are required to achieve the recommended minimum design life of 100 years

The designers of an extradosed bridge shall prepare a manual providing information about the basic demands on the components used in the bridge, the construction of the bridge, and suggestions about the inspection procedures of the bridge.

This Maintenance Manual shall be prepared after the completion of construction. Based on the design and construction, key areas for inspection need to be identified and the exact period between inspections for each component shall be included in the Manual. The Maintenance manual shall be approved by the client.

Special components such as the friction pendulum bearings and modular expansion joints will have a shorter cycle time than the main bridge structure due to their susceptibility to damage.

The maintenance manual shall contain at least the following:

(i) Reference to the Design Documents (inclusive of the Corrosion Protection Plan)

(II) Reference to the As-Built Records

(iii) Provide the specific timetable and procedures for inspection, testing, maintaining and record-keeping for the non-replaceable bridge components

(iv) Provide the specific timetable and procedures for inspection, testing, maintaining, replacing and record-keeping for the replaceable bridge components

(v) Indicate the qualification of the inspection and maintenance team required for each activity

(vi) Provide operational instructions and specifications of all mechanical and electrical equipment, if any

11.3.3 *Minimum Maintenance Requirements*

(i) The maintenance team needs to have a full set of drawings and maintenance schedules that indicate the frequency of inspection and maintenance for each part of the structure.

(ii) The maintenance plans should also highlight areas where something has gone wrong during construction and which need special monitoring.

(iii) For maintenance, repair, rehabilitation, and strengthening of various concrete components of the extradosed bridge IRC: SP: 40 shall be used.

(iv) The geometry of the extradosed bridge shall be surveyed at regular intervals, particularly after a major event, such as an earthquake, deep scour due to a major flood, etc. Anomalies in the geometry, if identified in a reliable survey, tend to reveal distress or defects which shall be thoroughly investigated in order that remedial actions can be formulated and taken.

(iv) Where scour poses a significant risk to the structure, a scour monitoring plan should be developed as a part of the bridge inspection and maintenance program. Inspection requirements, including method and frequencies, should be specified.

(iv) For maintenance, repair, and replacement of extradosed cable system guidance for extradosed cable manufacturer/supplier shall be sought.

11.3.4 *Bridge Management System*

A comprehensive scientific bridge management system consists of various tasks related to the collection of data for inventory, to period of inspection, details of inspection of each component and procedures to monitor, analyze and implement the activities related to maintenance works. Authorities can use the bridge management system to achieve optimum use of available funds, adaptation of bridges to traffic needs, and public safety.

Data Base Module

A pre-requisite for attempting any scientific system of management of bridge maintenance is to have a proper database module comprising inventory, inspection records, and maintenance, repair, and rehabilitation data.

Inventory

The inventory module consists of records of (i) administrative data (ii) general data (iii) technical data (iv) geometric data (v) hydraulic rata (vi) geotechnical data (vii) environmental data (viii) structural design data (ix) structural drawings, and (x) photographs of the bridge and various components before the bridge is opened to traffic.

Bridge inspection record

Bridge inspection record consists of data gathered and recorded while checking the condition of various spans and following up the deterioration process from one inspection to another. This could include (i) an initial inspection report, (ii) a routine inspection report (iii) a detailed inspection report, and (iv) a special inspection and or emergency inspection report.

Bridge maintenance, repair, and rehabilitation data file (MR&R)

This module is the collection of all the data pertaining to the details of each component of Bridge/Viaduct/ROB/RUB/Roads/Toll plaza/others, its inspection, its condition, etc. In addition to the basic design data and construction data, it contains maintenance, repairs, or improvement work data including as-built drawings, major repairs, and exceptional events such as earthquakes, abnormal floods, etc. The data as stored provides adequate information to make decisions for repair and strengthening works.

11.4 Structural Health Monitoring (SHM)

11.4.1 *Brief of Existing Guidelines for SHM*

There are many existing guidelines for SHM of structures in different countries like Canada, UK, European Union, China, Australia, Switzerland, and USA developed during the last two decades. Since the first structural health monitoring (SHM) guide, Guidelines for SHM [1], was released, new SHM standards have been added around the world. The Canadian intelligent sensing for innovative structures, which published SHM of Innovative Canadian Civil Engineering Structures, introduced basic techniques in SHM [2]. It also covers topics including repairing and strengthening of structures (use of fiber-reinforced polymers), intelligent sensing (integrated fiber optic sensors, fiber bragg grating sensors), economic significance (service life of structures will be extended with SHM), remote monitoring, and intelligent processing [2].

The European Union developed the structural assessment, monitoring, and control (SAMCO) association for the maintenance of infrastructure and disaster prevention [3]. The SAMCO database is mainly used for infrastructure maintenance, and especially for bridge management. The SHM in the UK is applied to different civil infrastructures (e.g., dams, bridges, offshore installations, nuclear installations, etc.). In the UK, dams were the first structures for which legislation mandated SHM inspections, though recently more attention has focused on bridges [4]. Recommendations for SHM implementation in the UK include low-cost system (investment) and high benefit (return). The implementation of sensors, i.e., the total quantity of sensors that will be put on a structure, their positioning, the distance between them, and the algorithms and processing should be appropriately determined beforehand to avoid data overloads, including a number of measurements, technical requirements, and different levels of accuracy for different sets of information collected. Environmental effects, including noise, on SHM systems could seriously influence or inhibit detection. In conclusion, the development of SHM in Europe provides good ideas and instructions for future SHM standards in other countries.

The rapid emergence of new standards and codes related to civil engineering in China has also led to the development of new SHM regulations [5]. More specifically, in 2016 the Chinese Ministry of Transportation published the first governmental SHM code regulating the installation of smart sensors in bridges [6]. China’s ability to develop and implement new civil engineering designs, while rapidly increasing the size and volume of structures, can serve as a model for transportation agencies in other countries in writing, approving, and implementing their own new SHM regulations [7,8].

Although the SHM standards from these institutions might not be enforced by their governments, the standards still give regulatory instructions for the implementation of SHM, which is supportive toward SHM code development.

11.4.2 *Need for SHM*

The factors affecting the static and dynamic behaviour of extradosed bridge on account of the hybrid nature of structural system, under the action of various applied loads, is determined by the interaction between following components [9] such as (i) flexural stiffness of the deck/girder superstructure, (ii) axial stiffness of the cable stay, (iii) height and longitudinal stiffness of the towers/pylons, (iv) lengths of adjacent spans and back-spans, (v) degree of fixity between the superstructure, the towers and the substructure and (vi) flexural stiffness of the main support piers.

The complexity in the design arising out of above factors can be evaluated by structural health monitoring of extradosed bridges.

Structural health monitoring is a non-destructive in-situ structural sensing and evaluation method that uses a variety of sensors attached to, or embedded in, a structure to monitor the structural response, to analyze the structural characteristics for the purpose of estimating the severity of damage/deterioration and evaluating the consequences thereof on the structure in terms of response, capacity, and service-life [10], [11], [2].

SHM can be classified as short-term and long-term monitoring and the situations in which they are adopted are given below.

Short-term monitoring consists of live load and dynamic testing. Data is only collected over a relatively short period of time when these field tests are taking place. The collected measurements can provide information about structural performance such as load and moment distributions, stress/strain levels, and serviceability issues. In addition, the collected data will provide valuable insights for future designs, and improve qualitative assessments of the bridge structures with different load tests.

Long-term monitoring usually consists of collecting ambient vibration and environmental data to evaluate structural condition over long periods of time. For instance, with long-term monitoring, changes or abnormalities in structural behavior can be detected without human interactions, or environmental effects (corrosion monitoring, seismic effects) on the performance of the structure can be quantified [12,13].

11.4.3 *Objectives*

The main objective of structural health monitoring could be any of the following or all

(i) design verification (short-term monitoring)

(ii) traffic management (long-term monitoring)

(iii) structural maintenance (long-term monitoring)

SHM is generally used to verify the design assumptions, to evaluate the performance of the bridge during construction as well as service, especially under extreme events, and when exposed to various environmental and loading conditions. This could be also used for the maintenance and /or traffic management of bridges.

Where design life is predicated on minimum conditions and performance of specific bridge components, such components shall be considered for instrumentation during the design stage.

11.4.4 *Overall Design Requirements Including Institutional Set up*

The complete design process of a SHMS can be summarized by following these steps:

1. Identify the need for monitoring
2. Set the objectives of the monitoring system
3. Undertake risk, uncertainty, and opportunity analysis
4. Identify parameters and response requirements

5. Select appropriate sensors
6. Select sensor location and communication system
7. Install the system and calibrate
8. Acquire data, process, manage and report
9. Assess data/study of the report
10. Decide on the need for remedial action

The design process covers several traditional study fields, such as civil engineering, electrical engineering, and computer science. Therefore, the Institute or the organisation undertaking the design and implementation of SHM should have bridge experts, electrical, electronics, and communication, and computer engineers. They should have trained staff for the installation and calibration of sensors. They should have a proven track record in the design, installation and performance monitoring of at least two long-span bridges.

Parameters, Location, and Number of Spans to be Monitored

While designing an SHM system, the first step is to decide whether short-term or long-term monitoring is required as discussed in Section 11.4.2.

The structural response parameters to be monitored include strain, acceleration and displacement of the deck at critical locations, cable forces, strain, and displacement of the tower tip.

The environmental parameters include wind intensity and direction, water level, atmospheric temperature, and humidity.

There are other loadings/ effect of actions to be monitored such as vehicular loading (traffic volume, type of vehicle, speed of vehicle, axle loading of trucks, axle spacing, and distance between trucks), pedestrian loading/ crowd loading, thermal effects, seismic effects and wind effects.

Selection of the location of sensors shall be done based on the three-dimensional analysis of extradosed bridge under various loadings. It may be appropriate to identify the critical locations for performance monitoring to be identified by the structural designers and included in the structural drawing.

The minimum number of spans to be instrumented and monitored shall be one unit (between two expansion joints) if the purpose of SHM is to verify the design assumptions and to collect baseline performance data.

In the case of multi-span extradosed bridges, with identical spans, at least one EDB unit shall be considered for SHM. All EDBs with main span length of more than 200m shall be selected for performance monitoring.

EDBs provided with seismic isolation bearings shall be instrumented to monitor the performance of the bridge under seismic action.

For more details on the design and functional requirements of SHM, the SHM frame work and safety evaluation of the bridge using the data collected are discussed in **Annexure-A.9**

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Annexure - A1

Salient Data about Extradosed Bridges in India

No.	Bridge Name	Owner / Contractor	Completion Year	Location in India	No of Traffic Lanes	EDB Bridge Length (m)	Span Arrangement (m)	Width (m) Total width	Width (m) C'Way Width	Number of Cable Planes	Presence of Exp Joint in mid span (Yes / No)	Girder Support on Pier (Integral / Bearings)	Cross Section of Girder (Concrete / Composite)	Girder Depth (m) at Pier	Girder Depth (m) Mid. Span	Tower Height above Bridge Deck level (m)	Tower Height above Foundation (m)
001	Veer Kunwar Singh Setu across river Ganga at Bhojpur (Arrah-Chapra Bridge)	BRPNNL S P Singla Construction	2017	Bihar	4	1920	60+15x120+60	20.5	15.0	1	YES, NB	Integral	PSC Box Girder	3.40	3.40	18.0	37.5
002	Re-construction of B.P.Mondal Bridge over river Koshi at Dumrighat	MoRT&H S P Singla Construction	2018	Bihar	2	290	75+140+75	12.68	11.59	2	No	Integral	PSC Box Girder	2.20	2.20	27.0	37.7
003	Bridge over river Hatania-Doania near Bakkhali	PW(Roads) Directorate, NH Wing S P Singla Construction	2019	West Bengal	3	340	85+170+85	14.8	10.5	2	No	Integral	PSC Box Girder	5.00	4.00	23.0	39.0
004	Sultanganj Bridge over river Ganga (Between Sultanganj and Aguwani Ghat)	BRPNNL S P Singla Construction	Under Construction	Bihar	4	1562.5	(125+2x162.5+125) x 2 (125+162.5+125) x 1	Varies from 25.5to	17	2	No	Integral	PSC Box Girder	5.50	4.00	24.0	51.0
005	Kacchi Dargah–Bidupur Bridge over river Ganga at Patna	BSRDCL DAEWOO – L&T JV	Under Construction	Bihar	3 lane + 3 lane with	9759.00	75+2x150+75 (Typ. Module)	32.4		1	YES, NB	Rigid	1 Cell with internal and	5.00	4.00	22.5	45.5
006	2nd Iswar Gupta Bridge Setu bridge across River Hoogly	WBHDCL Larsen & Tubro Ltd.	Under Construction	Kolkata, West Bengal	3 lane + 3 lane with	714	69+102+4x120+63	2x18		2	No	Rigid & Bearing	Single cell with external	3.00	3.00	22.0	43.5
007	Bridge across the Ganges on NH-31 Aunta-Simaria in the state of Bihar	NHAI S P Singla Construction	Under Construction	Bihar	6	1865	70+15x115+70	34	2x13	1	Yes	Integral	PSC Box Girder	3.75	3.75	17.0	38.272
008	Nivedita Setu over river Hooghly	NHAI / SVBTC Larsen & Tubro Ltd.	2007	West Bengal	6	880.00	55+7*110+55	28.6	2 x 12.5	1	YES, NB	Rigid	Single cell with internal	3.20	3.20	14.0	43.34
009	Extradosed Bridge across Durgham Cheruvu lake, Hyderabad	GHMC Larsen & Tubro Ltd.	2020	Hyderabad, Telangana	2 lane + 2 lane with	427.51	97.66+233.85+96	25.5		1	No	Rigid	Single cell with internal	7.819	5.819	28.0	51.6 / 54.8
010	Goa Bridge	Goa State Infrastructure Development Corporation Limited	2004	Goa	2 lane	180m	100m+80m	13m	---	Double	No	---	Composite	---	---	---	---
011	Barapulah Bridge over river Yamuna Larsen & Tubro Ltd.	DT&TDC Larsen & Tubro Ltd.	Under Construction	New Delhi	(3 lane+ footpath+	552.50	85+3x127.5+85	20.8		2	YES, NB	Rigid	3 cell	4.00	4.00	17.0	35.78
012	New four lane Extradosed Bridge across River Narmada (3rd Narmada Bridge)	NHAI Larsen & Tubro Ltd.	2017	Bharuch, Gujarat	4 lane+ footpath	1344	96+8x144+96	20.8		2	YES, CHB	Rigid	3 cell	4.00	4.00	18.0	38.5
013	Mumbai Metro-WEH		2013	Mumbai	Metro Railway	132	2x23+86+2x23	11,75	8,85	2		bearing	U shape			19.1	38.04
014	Moolchand Crossing	DMRC	2010	New Delhi	Double Track	167.5m	51m+65.5m+51m	9.36	---	Single	No	Integral	Concrete	---	---	---	---
015	Pragati Maidan	DMRC	2006	New Delhi	Metro Railway	196.35	24,8+31,25+93+24,8+22,5		9,36	2		rigid	U shape			12,43	
016	New 6 lane bridge over river Brahmaputra connecting Guwahati with North Guwahati	Govt. of Assam, PWRD S P Singla Construction	Under Construction	Guwahati	6	1240.00	120 + 5 x 200 + 120	33.0	25.0	1	No	FPB	PSC Box Girder	4.25	4.25	32.50	57.0
017	New MG Setu over river Ganga at Patna	MoRT&H S P Singla Construction	Under Construction	Bihar	4	5566.00	7 x (121+242*2+121) + 1 x (121+242+121)	21.5	16.5	2	No	Integral	PSC Box Girder	7.00	4.00	36.0	65.0
018	CROSS OVER SWAP-01 NEAR LALIT BHAWAN, PATNA	BRPNNL S P Singla Construction	Under Construction	Bihar	2	193.00	50+93+50	10.4	7.5	2	No	Integral	PSC U-trough	2.50	2.50	9.0	19.0
019	CROSS OVER SWAP-03 ON SERPENTINE ROAD, PATNA	BRPNNL S P Singla Construction	Under Construction	Bihar	2	100.00	2 x 50	8.9	6	2	No	Integral	PSC U-trough	2.50	2.50	8.4	18
020	Bridge over river Brahmaputra between Dhubri & Phulbari	NHIDCL	Under Bidding	Assam	6	12612.3	34 x (61.825+123.65 x 2 + 61.825)	28.0		1	YES						
021	New Bridge over river Ganga at Bhagalpur	BRPNNL S P Singla Construction	Under Construction	Bihar	4	4301.6	(57.7+115.4x3+57.7)x4+(57.7+121+126.6x2+63.3)+(63.3+126.6x3+63.3)+(63.3+126.6+135.3+120+60)+(49+109+142+120+104.4+64.8)	29.5	2x10.5	1	Yes	Integral	Concrete	Varies	3.75	24.0	42
022	GANGA BRIDGE AT PRATAPGARH, U.P	SPSCPL - ATEPL JV	2022	Uttar Pradesh	2	1272	(87+144+87)X4	11.6	7.5	2	Yes	Integral	Concrete	3.75	3.75	17.9	34.12
023	Bridge across river Ganga on NH-31 at Shri Rampur Ghat, Balia	UPSBC Ltd. ATEPL-SPS JV	2018	Uttar Pradesh	2	2544	(87m+144m+87m) x 8	11.3	7.5	2	No	Integral	Concrete	3.75	3.75	18.0	41.64
024	Extradosed Bridge across River Mahi at Dungarpur	PWD, Rajasthan S P Singla Construction	Under Construction	Rajasthan	2	576m	72 m+ 3 x 144m + 72m	16.245	12.0	2	YES, NB	Integral + Bearing	Single cell box	5.00	3.00	14.84	26.0
025	Ganga Bridge on Sahibganj Bypass	MoRTH Dilip Buildcon	Under Construction	Jharkhand	4	6000	15x(110+180+110)	28.0	2 x 10.0	1	NO	Integral	Single cell box with	4.75	4.75	23.0	45.5
026	Siddapur extradosed bridge	KRDCL	2006	Karnataka	2	56.00	28+56+28	10.5	9,5	2	NO	On bearings	PSC Box girder	2.20	2.20	5,275	
027	Bridge Across Sharavathi Backwaters, Karnataka	MoRT&H M/S Dilip Buildcon Limited - SRBG Ltd (JV)	Under Construction	Karnataka	3	740.00	(104.5m+3x177m+104.5m)	16.0	11.0	2	NO	Integral / Bearings	PSC Box girder	5.50	3.25	38.5	61.2
028	Bridge over River Ganga at Phaphamau, Prayagraj	MoRT&H S P Singla Construction	Under Construction	Uttar Pradesh	6	860.00	130+200+200+200+130	33.0	2 x 14	1	NO	Integral / bearing	PSC Box girder	7.00	5.00	36.65	55.75

Annexure - A2

Example for preliminary Design of an Extradosed Bridge

(Clause no 5.4, Chapter 5)

This Appendix includes a worked-out example to Illustrate necessary steps in the design of an extradose bridge. The procedure generally covers preliminary dimensioning, loads and load combinations, design checks for the extradose cables, and flexure check for the box girder deck. Other checks viz Shear, Torsion for the longitudinal design of superstructure not covered in this example. The example presented below explains one of the several ways of analysis of an extradosed bridge superstructure. Designer may choose their own analysis method to comply with design requirements of the guideline.

1. General Arrangement

The general arrangement drawing of the extradosed bridge should be based on the functional requirement, clearances, span based on site constraints, and construction methodology.

Consider an example of a two-lane extradosed bridge

a) Span arrangement - 72m +144m+72m

b) The pylon height above deck up to at top cable centre is about 0.1L (L – Span) i.e., 14m; hence the total height of the pylon is kept 15m.

c) The depth of the superstructure at pylon is 5m and at mid-span is 3.5m

d) Bridge shall have two planes of cables.

2. Materials Properties

Grade Concrete:

Superstructure	: M60
Pylon	: M60
Reinforcement	: Fe500
HTS for Prestressing	: 1860 Mpa
Extradose Cable	
Nominal Breaking Strength	: 1860Mpa
Strand type	: 15.7mm dia, 150mm^2 Area, 7 ply low relaxation, Galvanised.
F_{GUTS}	: 1860Mpa

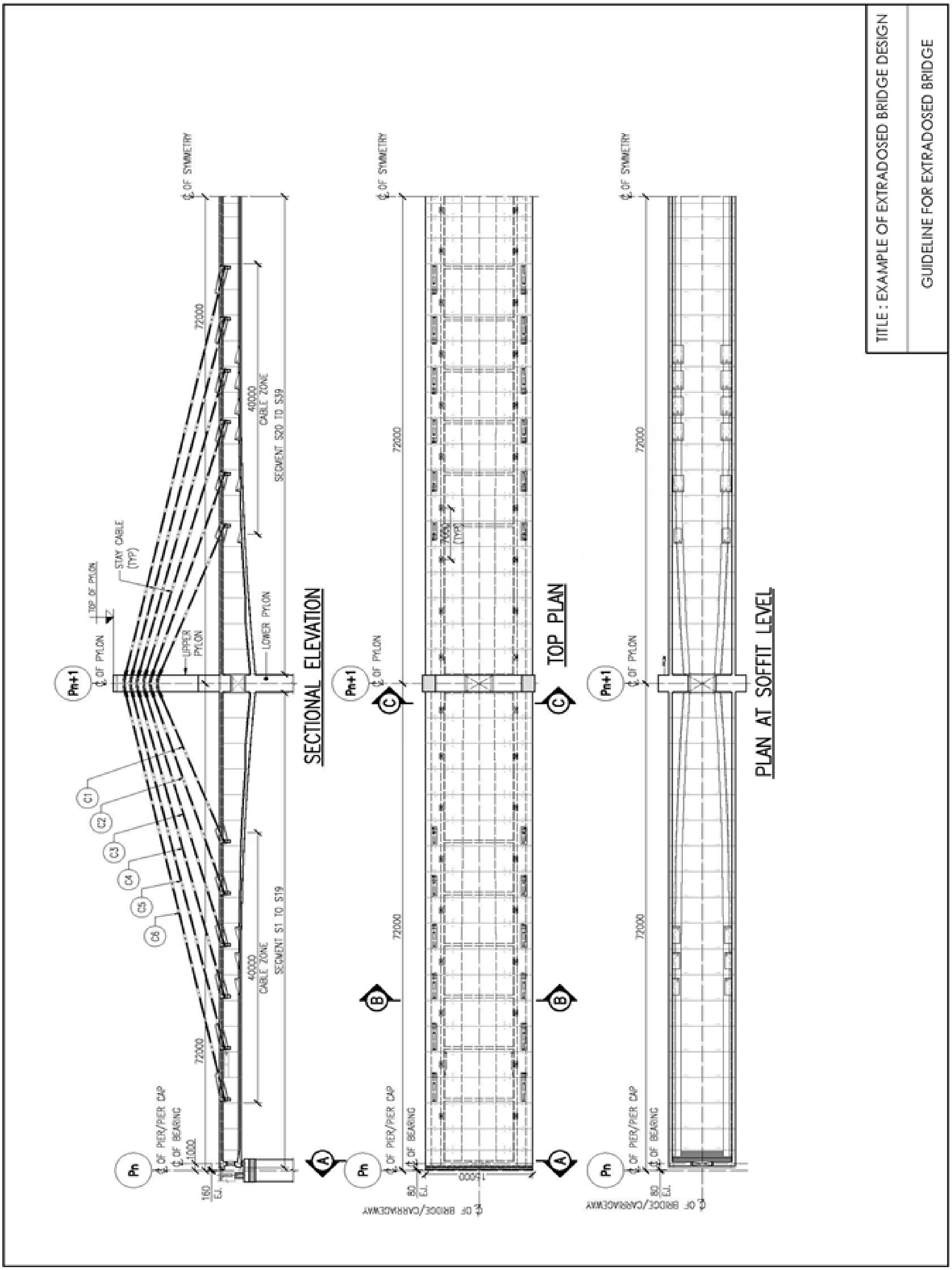


Fig A2-1 Dimensions / Segmentation of bridge superstructure

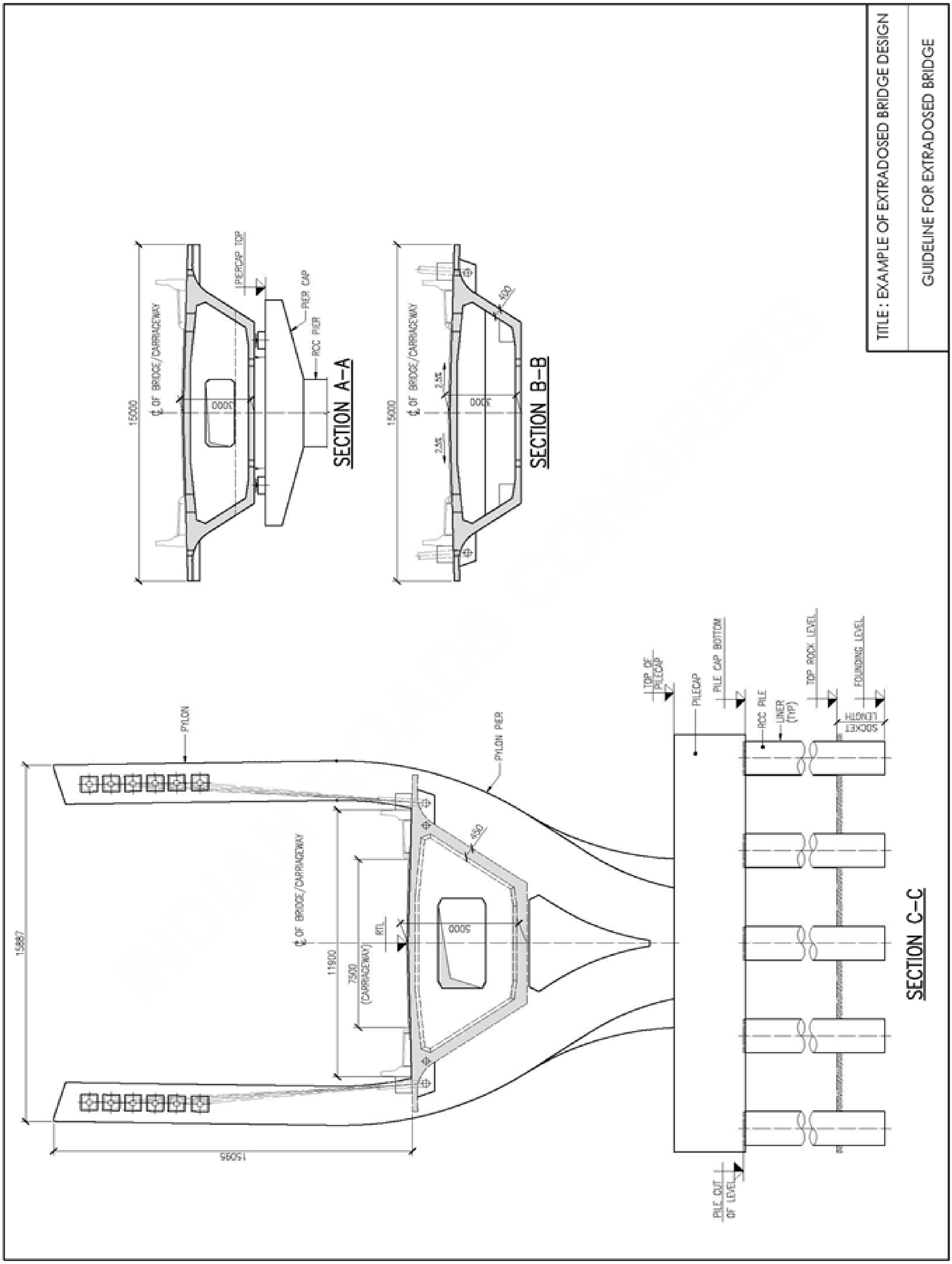


Fig A2-2 Cross Section of bridge superstructure

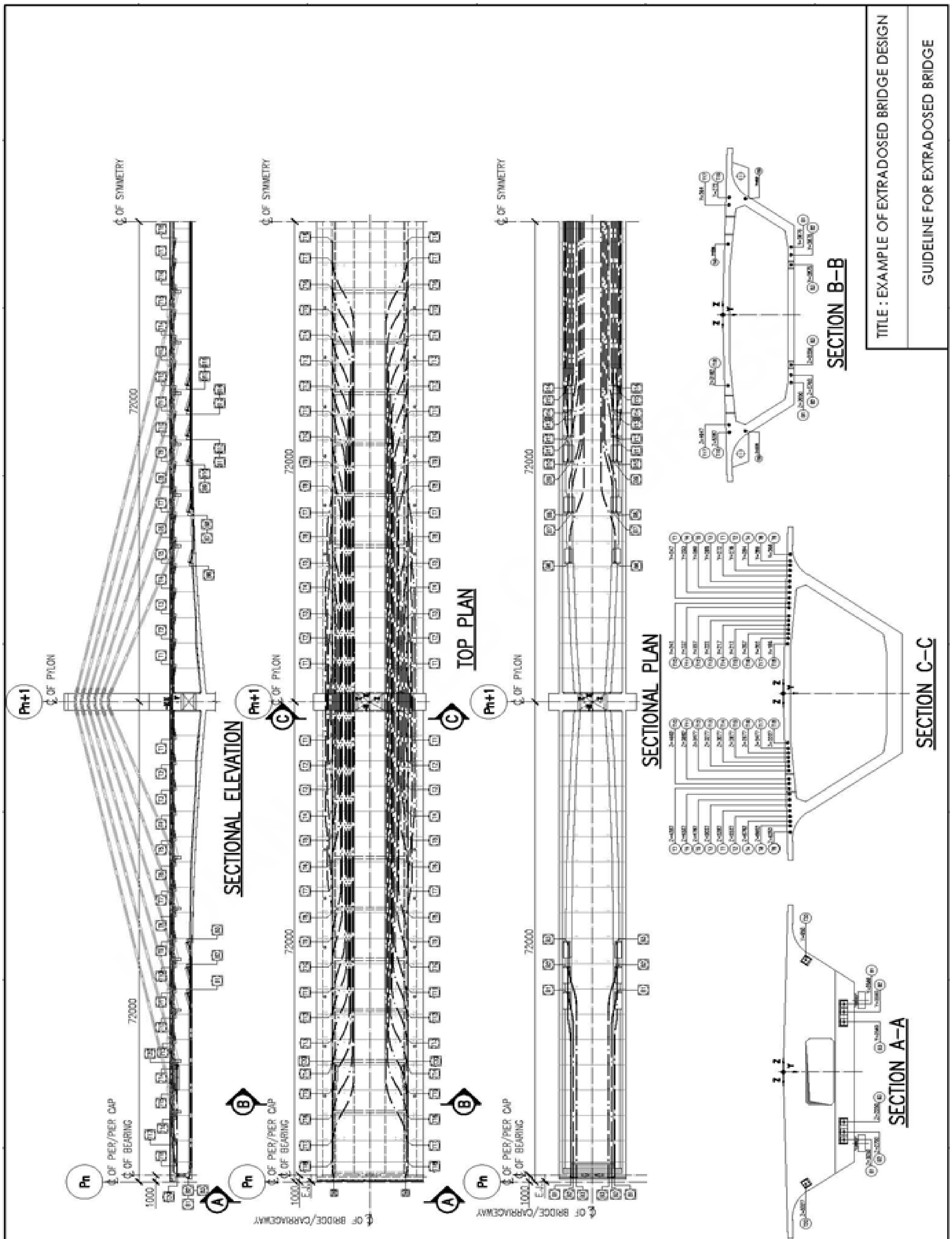


Fig A2-3 Prestressing Layout

3. Load calculation:

- *Dead Load*

  Dead load of the box girder :

  At Support : 313 kN/m

  At Mid Span : 231 kN/m

  Weight of Blisters : 10kN /no

  Weight of the cross beam at Cable : 120 kN / no

- *Superimposed dead load (SIDL)*

  Crash Barrier : 20.35 kN/m

  Services : 2.0 kN/m

  Footpath / Kerb / Railing : 10 kN/m

  Wearing coat (Surfacing) : 10.9 kN/m

  Total SIDL : 43.25 kN/m

- *Vehicular Live Load (LL)*

  Live load on carriageway shall be as per IRC: 6 – 2017

  Since it is a 2-lane bridge critical of following load case shall be considered for design

  - 2 Lanes of Class A wheeled vehicle on each carriage way.
  - 1 Lane of Class 70R wheeled vehicle
  - Special vehicle at the center of the carriageway

- Footpath Live Load : 2.06 kN/m^2 x 1.5 x 2 = 6.18kN/m

4. Preliminary sizing of the extradose cables

No of cable on each side of pylon : 12

Average inclination of the cable : 15.5 deg

Length of the load to carried by cable : 40 m (Length of cable zone)

Total load to carried by cable (DL+SIDL) : (231 + 43.25) x 40
(considering only per meter weight of mid segment)

: 10970 kN

Force along the cable : 10970/sin (15.5)

: 41049 kN

Force in each cable due to dead load and SIDL : 41049/12 = 3420 kN

Adding 20% for Live Load and Temperature loads : 3420 x 1.2 = 4104 kN

Preliminary Sizing of the cable is based on the above force. If permissible stresses in the cable are considered as 0.6 F_{GUTS}, the approx. number of strands required in each cable will be 24 numbers of 15.7mm dia (Approx. 14kg/sqm). A preliminary analysis of the bridge with these sizes shall be carried out to verify the sizing.

Above method indicated is the one of the numerous possibilities of sizing of the cable for design of extradosed bridges. Designer can choose his own method and can arrive at different sizing.

5. Analysis Model

Analysis model is prepared in software as shown below. Analysis model of 288m length consists of the Box girder/ pylon modelled with beam element. Extradosed Cable is modelled with cable element. The Cables are in two planes and are connected to the box girder with rigid link with actual eccentricity of the cable anchorage w.r.t centroidal axis of the superstructure.

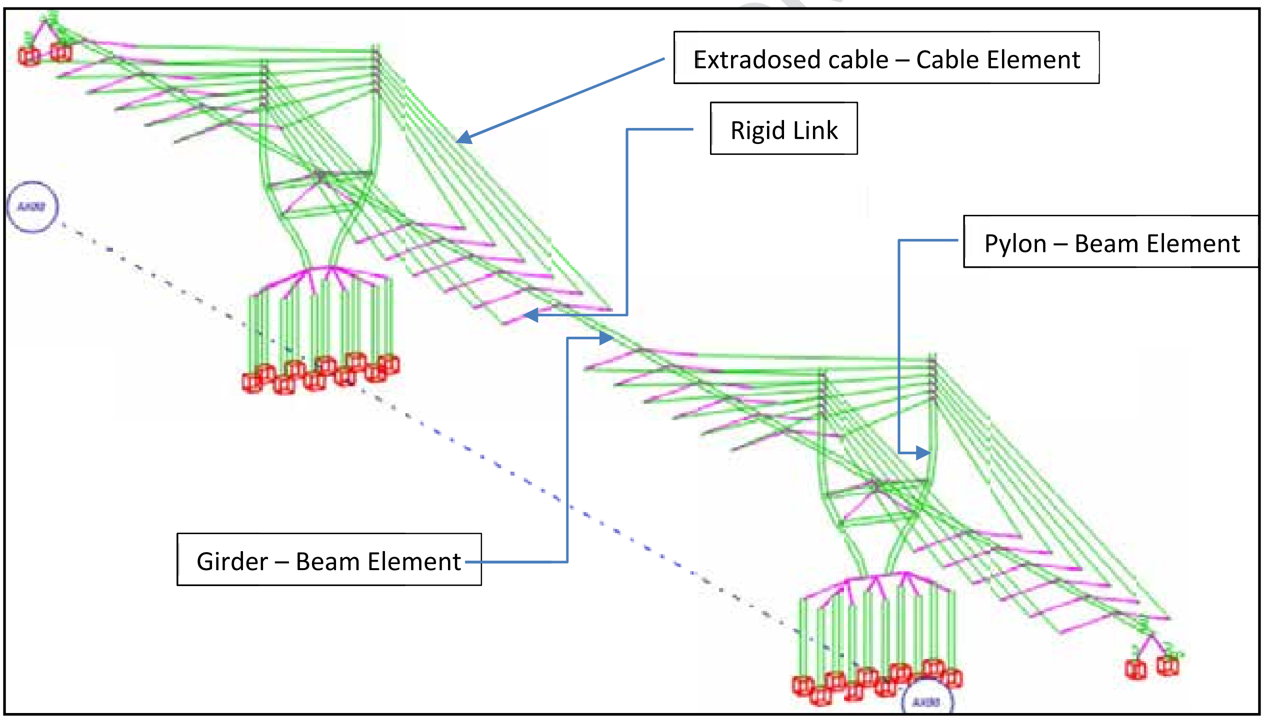


Fig A2-4 Analysis Model

6. Construction Stages in Analysis

Superstructure is proposed to be constructed by cast-in-situ, segmental construction. Fig A2-1 shows segmentation of the superstructure. The typical stages of construction are shown below in sketch A2-5. In the analysis model considers construction staged analysis which includes prestress, long term material behaviour like creep and shrinkage as per the age of the concrete, permanent and construction loads due to travelling form etc.

Following construction stages are considered in the analysis.

Construction Stages

CS	Type	t [d]	RH [%]	T [°C]	laun_1 [m]	laun_2 [m]	Designation
1	G_1						Erection of Pier
2	G_1						Erection of Pylon
3	G_1						Casting of Pier Head on Staging
22	P						Prestress Stg1
23	G_1						Seg Wt Activation
24	C	3	50	20			CS
30	B						TF Self Wt
31	G_1						Segment Casting
32	P						Prestress (2)
34	G_1						Seg Wt
35	C	3	50	20			K creep step
36	G_1						Stay Activation
37	G_1						Stay DL
38	G_1						Stay Stressing
40	B						TF Self Wt
41	G_1						Segment Casting
42	P						Prestress (3)
44	G_1						Seg Wt
45	C	3	50	20			K creep step
46	G_1						Stay Activation
47	G_1						Stay DL
48	G_1						Stay Stressing

Steps 49 to 183 … Repetition of Cycle for Segment 4 to Segment 17

184	G_1						Seg Wt
185	C	3	50	20			K creep step
186	G_1						Stay Activation
187	G_1						Stay DL
188	G_1						Stay Stressing
190	B						TF Self Wt
191	G_1						Segment Casting
192	P						Prestress (18)
194	G_1						Seg Wt
195	C	3	50	20			K creep step

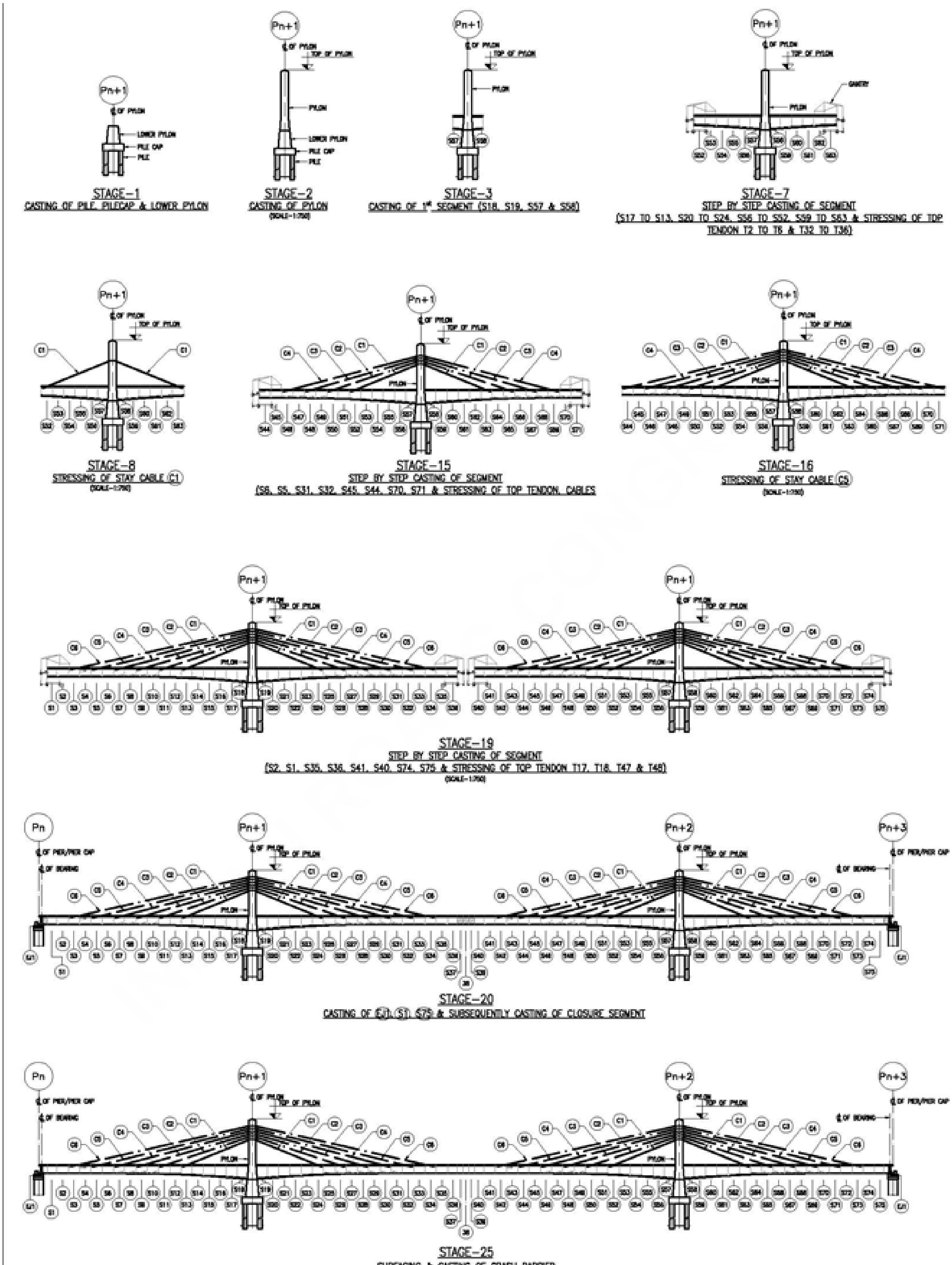


Fig A2-5 Construction sequence for superstructure

196	G_1						Stay Activation	
197	G_1						Stay DL	
198	G_1						Stay Stressing	
211	G_1						EJ Segment Activation Both Side	
212	G_1						EJ Segment DL Activation Both Si	
250	P						P prestress	
252	C	3	50	20			K creep step	
255	P						P prestress	
257	C	3	50	20			K creep step	
260	P						P prestress	
262	C	3	50	20			K creep step	
265	P						P prestress	
267	C	3	50	20			K creep step	
291	C	7	50	20			K creep step	Bottom Prestress
300	P						P prestress	on Side Span
302	C	0	50	20			K creep step	
305	P						P prestress	
307	C	0	50	20			K creep step	
310	P						P prestress	
312	C	0	50	20			K creep step	
315	P						P prestress	
317	C	0	50	20			K creep step	
321	C	7	50	20			K creep step	
330	B						TF Self Wt	
331	G_1						Seg 21 Casting	
332	G_1						Seg 21 Wt	
340	B						TF Self Wt	
341	G_1						Closure Seg casting	
342	G_1						Closure Seg Wt	
345	C	3	50	20			K creep step	
350	P						P prestress	
352	C	0	50	20			K creep step	
355	P						P prestress	
357	C	0	50	20			K creep step	
360	P						P prestress	
362	C	0	50	20			K creep step	Bottom Prestress
365	P						P prestress	on Central Span
367	C	0	50	20			K creep step	
370	P						P prestress	
372	C	0	50	20			K creep step	
375	P						P prestress	
377	C	0	50	20			K creep step	
501	G_2						SIDL_Surfacing	
502	G_2						Crash FP	
503	G_2						Stg 2	
505	C	100	50	20			K creep step	
506	C	100	50	20			K creep step	
510	C	500	50	20			cs	
520	C	1000	50	20			cs	Long Term Creep
530	C	3024	50	20			cs	Shrinkage
531	C	8129	50	20			cs	
532	C	21847	50	20			cs	

7. *Load Combinations for Extradose Cables*

Extradosed cable shall be checked for SLS combination as per Chapter 7 of this guidelines

SLS combination for Extradosed cable

Load Case	DL	SIDL	WC	LL*	WL	T
Rare Comb (LL leading+WL)	1.0	1.0	1.2	1.0	0.6	0.0
Rare Comb (WL leading+LL)	1.0	1.0	1.2	0.75	1.0	0.0
Rare Comb (LL leading+T)	1.0	1.0	1.2	1.0	0.0	0.6
Rare Comb (T leading+LL)	1.0	1.0	1.2	0.75	0.0	1.0
Rare Comb cable Replacement (LL leading+T)	1.0	1.0	1.2	1.0	0.0	0.6
Rare Comb cable Replacement (T leading+LL)	1.0	1.0	1.2	0.75	0.0	1.0

*LL Reduced Live load in case of cable replacement case

Summary of Forces in cable

						Veh. Live load				
Cable no	**Dead Load**	**Surfacing**	**CB FP**	**Stage 2 Stressing**	**Creep / Sh**	**Max**	**Min**	**FPLL**	**Wind**	**Temp**
	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]
C1	2407.3	26.7	89.1	1051.6	-164.2	66.8	-2.5	10.9	25.9	121.4
C2	2540.2	27.2	90.6	954	-144.0	66.7	-2.2	11.0	26.3	137.7
C3	2677.4	26.8	89.3	863.9	-121.5	67.0	-2.4	10.9	25.9	154.0
C4	2778.9	25.6	85.4	778.2	-100.3	67.2	-5.0	10.4	24.8	169.5
C5	2849.4	23.9	79.5	694.4	-80.6	66.9	-9.6	9.7	23.1	183.5
C6	2879.6	21.8	72.7	510.4	-63.3	69.8	-17.4	8.9	21.1	195.5

Check for $\Delta\sigma_{sls}$ (Change in force in the cable with freq. LL)

Cable no	No of Strands	Area of Cable	Δ_{sls}	Check
		mm^2	(Mpa)	
C1	24	3600	13.40	< 50, Ok
C2	24	3600	13.44	< 50, Ok
C3	24	3600	13.46	< 50, Ok
C4	24	3600	12.96	< 50, Ok
C5	24	3600	11.94	< 50, Ok
C6	24	3600	10.92	< 50, Ok

Check for f_{sls}

Cable no	No of Strands	Area of Cable	Comb 1	Comb 2	Comb 3	Comb 4	0.6 F_{GUTS}	Check
		mm^2	kN	kN	kN	kN	kN	
C1	24	3600	3509.1	3516.7	3566.4	3612.2	4017.6	OK
C2	24	3600	3566.9	3574.7	3633.8	3686.1	4017.6	OK
C3	24	3600	3634.7	3642.3	3711.6	3770.4	4017.6	OK
C4	24	3600	3665.4	3672.7	3752.2	3817.4	4017.6	OK
C5	24	3600	3661.8	3668.7	3758.1	3829.1	4017.6	OK
C6	24	3600	3516.9	3523.1	3621.6	3697.5	4017.6	OK

ULS combination for Extradosed cable

Load Case	DL#	Prestress In stay cable	SIDL	WC	LL**	WL	Seismic	Cable Break
Basic Comb (LL leading + WL)	1.35	1.0	1.35	1.75	1.5	0.9	-	0.0
Basic Comb (WL leading + LL)	1.35	1.0	1.35	1.75	1.15	1.5	-	0.0
Accidental Comb	1.00	1.0	1.0	1.0	0.75	0.0	-	1.1
Seismic Comb	1.35	1.0	1.35	1.75	0.2	-	1.5	-

** LL - No special vehicle and congestion factor to be consider for accidental combination

DL# – Dead load action as per cantilever construction analysis model including Stay forces.

Check for F_{ULS}

Cable no	No of Strands	Area of Cable	Comb 1	Comb 2	0.8 F_{GUTS}	Check
		mm^2	kN	kN	kN	
C1	24	3600	3957.0	3938.1	5356.8	OK
C2	24	3600	4005.2	3984.7	5356.8	OK
C3	24	3600	4052.5	4030.5	5356.8	OK
C4	24	3600	4053.6	4030.8	5356.8	OK
C5	24	3600	4010.4	3988.5	5356.8	OK
C6	24	3600	3818.7	3798.4	5356.8	OK

8. Check for Superstructure

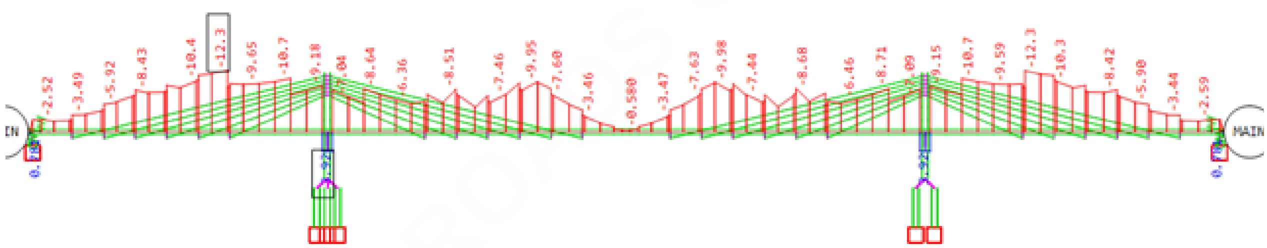
8.1. SLS Check

8.1.1. Combinations of superstructure

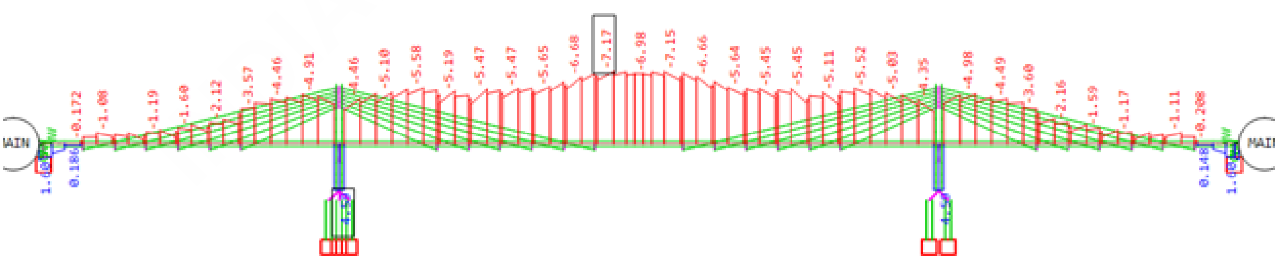
Load Case	DL	SIDL	WC	LL*	WL	T	
Frequent Comb (LL leading + WL)	1.0	1.0	1.2	0.75	0.5	0.0	Check 0.2mm crack width
Frequent Comb (WL leading + LL)	1.0	1.0	1.2	0.2	0.6	0.0	
Frequent Comb (LL leading + T)	1.0	1.0	1.2	0.75	0.0	0.5	
Frequent Comb (T leading + LL)	1.0	1.0	1.2	0.2	0.0	0.6	
Rare Comb (LL leading + WL)	1.0	1.0	1.2	0.75	0.5	0.0	Check for Max. Compressive stress
Rare Comb (WL leading + LL)	1.0	1.0	1.2	0.2	0.6	0.0	
Rare Comb (LL leading + T)	1.0	1.0	1.2	0.75	0.0	0.5	
Rare Comb (T leading + LL)	1.0	1.0	1.2	0.2	0.0	0.6	

*LL Reduced Live load in case of cable replacement case

8.1.2. Stress check for above load combination



Stresses at Bottom of the concrete (Frequent Comb)



Stresses at Top of the concrete (Frequent Comb)

Stresses in the concrete at all the section under frequent load combinations are compressive except at ends for which crack width checks shall be carried out. Similarly check for maximum compressive stresses in concrete under rare combinations of loading shall be carried for SLS check as per IRC 112.

8.2. Strength check of superstructure (ULS)

Load Case	DL#	Prestress in Extra-dosed Cable	SIDL	WC	LL	WL	T	Cable Break
Basic Comb (LL leading + WL+T)	1.35	1.0	1.35	1.75	1.5	0.9	0.9	0.0
Basic Comb (WL leading + LL+T)	1.35	1.0	1.35	1.75	1.15	1.5	0.9	0.0
Basic Comb (T leading + LL+WL)	1.35	1.0	1.35	1.75	1.15	0.9	1.5	0.0
Accidental Comb	1.35	1.0	1.0	1.0	0.75	0.0	0.5	1.1

#DL – Dead load action as per cantilever construction analysis model including Stay forces.

8.2.1. Check for ULS (By Standard Method of the Guideline)

Software Output

```
Extradosed Span
ULS SUPERSTRUCTURE

Combinations of Load Cases
 9101 ( CS  633) ULS-Dead+sidl+P+4L-LL
       MAX  +  My   :
                1.00 * G_1 + 1.35 * G_2 + 1.00 * P + 1.50 * L_1 + 1.50 * L_3
                0.25 * T_5 + 0.90 * W + 0.35 * G_5
 9102 ( CS  633) ULS-Dead+sidl+P+4L-LL
       MIN  +  My   :
                1.00 * G_1 + 1.35 * G_2 + 1.00 * P + 1.50 * L_1 + 1.50 * L_3
                0.25 * T_5 + 0.90 * W + 0.35 * G_5
```

Maximum BM and axial force in member are calculated on section at face of pylon as
Normal Force – 48467 kN (Compressive)
Bending Moment – 233248 kNm (Hogging)

8.2.2. Section Check

Section check is carried out in Sofistik, capacity of the section is calculated for above forces and presented below.

Beam	x[m]	SNo	LC	NRd	MyRd	MzRd	ε-1	ε-2	γ-c	γ-s	rel	As	Lay.
				[kN]	[kNm]	[kNm]	[o/oo]	[o/oo]	[-]	[-]	[-]	[cm2]	
				ΔNi	ΔVyi	ΔVzi	yn	zn	e+	e-	z		
				[kN]	[kN]	[kN]	[mm]	[mm]	[mm]	[mm]	[mm]		
				Designation		ε-o	ε-min	ε-max	τ-b	σ-min	σ-max		
				shear cut		T/Tmax	D/Dmax	Z/Zmax	N[kN]				
20001	0.000	107	9102	-84576.7	-407021.	-342.23	-3.50	0.97	1.50	1.15	1.75	537.60	Z
									rel(M-live)[1]		4.82		
					-0.00	111.86	-9999	1200	1862.-	1679.	4359.		
				Material	7		-3.50	1.22		-25.5	0.00		
				Tendons	3		6.66	7.91		1299.	1395.		
			9102	tendon	101					1201.	1299.		
				tendon	102					1307.	1393.		
				tendon	103					1322.	1394.		
				tendon	104					1336.	1394.		
				tendon	105					1345.	1394.		
				tendon	106					1359.	1395.		
				tendon	107					1356.	1395.		
				tendon	108					1350.	1394.		
				tendon	109					1346.	1394.		
				tendon	110					1333.	1394.		
				tendon	111					1334.	1394.		
				tendon	112					1320.	1394.		
				tendon	113					1320.	1394.		
				tendon	114					1305.	1394.		
				tendon	115					1304.	1394.		
				tendon	116					1288.	1393.		
				tendon	117					1287.	1393.		
				tendon	118					1274.	1393.		

Moment of resistance as per PT tendons provided in section– 407021kN (Hogging) for Axial force of 84576kN which is relative capacity of 1.75 hence ok

Other sections of the structure viz pylon, diaphragm shall be designed in line with IRC 112. Once all the section are designed and checked for all the stages i.e., Construction stage and at Service, pre-camber analysis shall be carried out to calculate the required camber to be provided in superstructure and pylon during the construction.

Annexure A3

Approach to Design of Stay Cable and Extradosed Cables as per International Guidelines

(Chapter 7)

This Annexure presents philosophy and Methods for the design of cable for cable stayed bridges and extradosed bridges adopted by Japanese Code *"Specifications for Design and Construction of Cable Stayed Bridges and Extradosed Bridges"*, Setra – Stay Cable Recommendation and PTI Recommendations which are the codes available for design cable supported bridges. Also, recommendation of the FIB 89 are presented at the end.

I. ***Specifications for design and construction of cable stayed bridge and extradosed bridge- JPCI***

As per Kasuga in his paper [1], when designing structures that are reinforced using stays, rather than defining in advance whether the bridge will be a cable-stayed bridge or an extradosed bridge and then determining the allowable stress for the stays, the more rational approach would be to design the stays by focusing on the stress change caused by live loads that affect fatigue. This would make it possible to design each stay separately and enable the allowable stress to be set individually for each stay. From the outset, unlike suspension bridges, the stress change on a cable stayed bridge will differ depending on the stays, so it is not rational to define the allowable stress using a single value of 0.4 f_{pu}. This knowledge is reflected in JPCI specifications.

The specification allows two kinds of design method. One is normal fatigue design using fatigue load and design lifetime of a bridge (Method A). However, it is usually difficult to estimate the amount of future traffic and heavy trucks, especially in local roads. In that case, another approximation method using stay-cable stress change owing to designed vehicular live loads is introduced (Method B). Figure A3-1 shows the comparison of Methods A and B. Figure A3-3 shows the design flow for stay-cable design.

As per Method B the stress variation in the cable due to design live load $\Delta\sigma_L$ is determined, based on $\Delta\sigma_L$, permissible stresses in cables at service loads is determined to carry our serviceability check. The above relation between the stress changes due to live and permissible stress is given for both prefabricated wire type and strand type cables.

Method B is defined such that it ensures adequate safety in comparison with bridges designed using Method A.

Figure A3-1 shows fatigue stress range ($\Delta\sigma_{2E6}$) in the cable (Based fatigue design performed for 3 bridges). This fatigue stress range is compared to fatigue strength (f_{scrd}) divided by safety factor (y_b).

For a strand stay cable fabricated on site using wedges, based on a system with fatigue strength of at least 120 N/mm^2 at 0.6fpu or at least 200 N/mm^2 at 0.4f_{pu}. In this situation, g_b is 1.4. The shaded section of the figure is the range determined by Method A (Δs_{2E6}) with the fatigue design conditions indicated above. This is two-thirds of the $\Delta\sigma_L$, as prescribed by Method B, there is a safety factor of around 2.0 with respect to f_{scrd}/y_b.

For a galvanised wire stay cable made at a factory as a cold-cast cable or as a cable with buttonhead anchorages, the relationship between the $f_{scrd}/y_{b.}$ and the $\Delta\sigma_{2E6}$ estimation line is shown in Figure A3-1, based on a system with fatigue strength of at least 180 N/mm2 at 0.6fpu or at least 230 N/mm2 at 0.4fpu, and like the figure for the cable fabricated on site. It can be seen from the figure that the factory-made cable also has a safety factor of around 2.0 with respect to $f_{scrd}/y_{b.}$

Therefore, in the stays designed by Method B, Δ_{sL} is determined to require a safety factor of about 2.0 for $\Delta\sigma_{2E6}$ with respect to $f_{scrd}/y_{b.}$

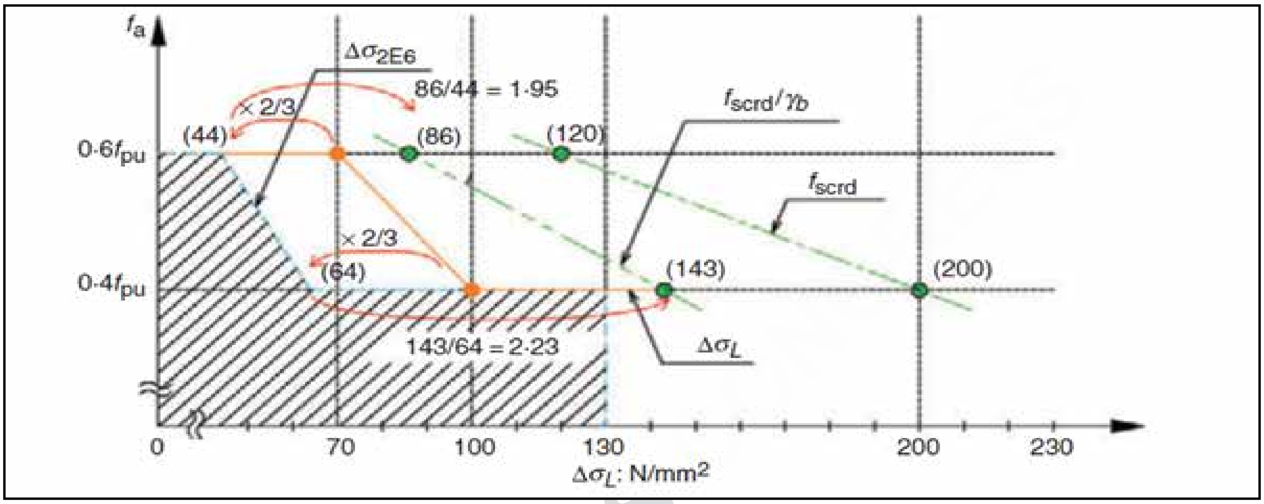


Figure A3-1 : Comparison of Method A and Method B

According to this specification designer can choose the tensile stress in each stay cable from $0.4f_{pu}$ to $0.6f_{pu}$ (Figure A3-2) continuously and not one value of tensile stress in one bridge.

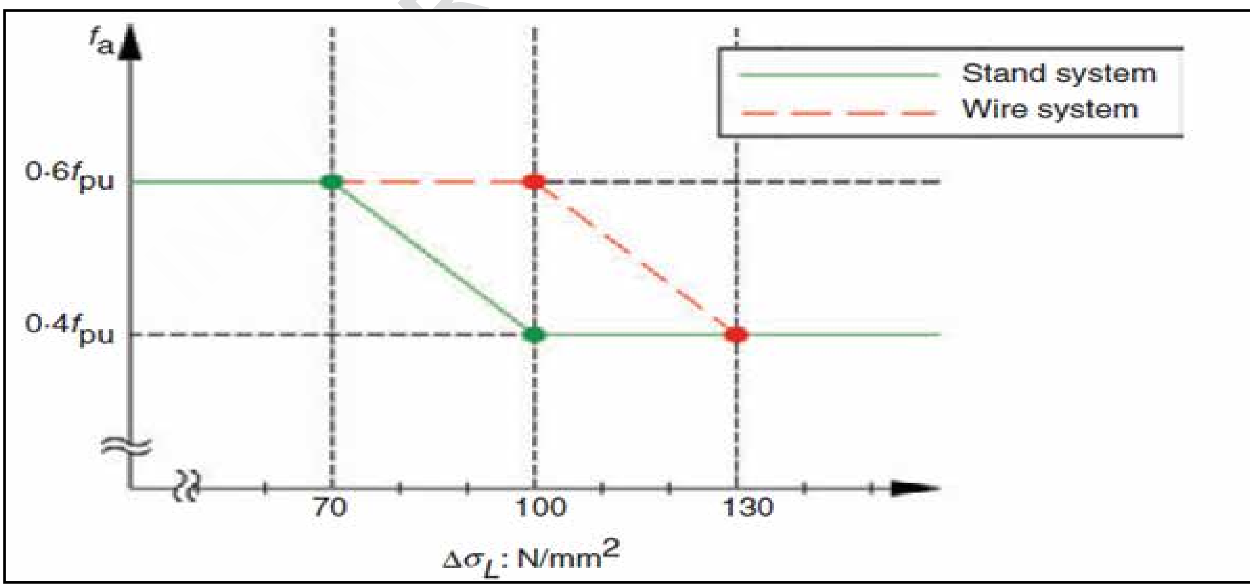


Figure A3-2: Allowable stress in SLS to Design LL stress variation

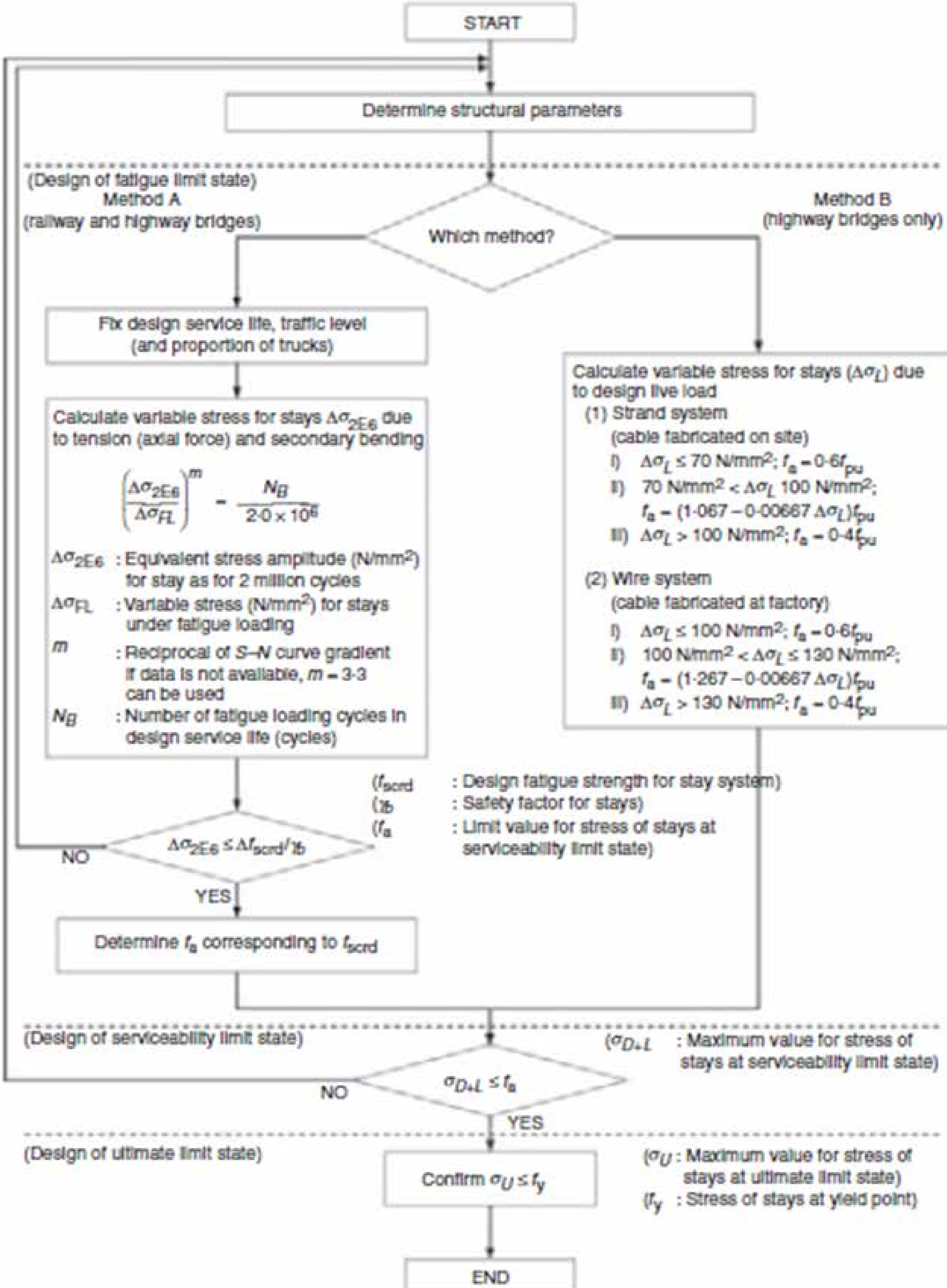


Fig: A3-3: Flow Chart for Stay Cable design as per JPCI Specifications

II. *Setra Recommendation - Cable Stays*

The SETRA recommendations relates the allowable axial stress σa at SLS for the stay cables with the stress variation ∆σL due to SLS live loads and can be determined from the following equation. The allowable axial stress σ_a is applicable for the entire stay cable system [SETRA 2002] unlike JPCI recommendation which applies to individual cables.

$$\sigma_a = \begin{cases} 0.6\,\sigma_{UTS} & \text{if } \Delta\sigma_L \leq 50\,\text{MPa} \\ \sigma_a \leq 0.46\left(\dfrac{\Delta\sigma_L}{140}\right)^{-0.25}\sigma_{UTS} & \text{if } 50 < \Delta\sigma_L \leq 140\ \text{MPa} \\ 0.46\,\sigma_{UTS} & \text{if } \Delta\sigma_L > 140\,\text{MPa} \end{cases}$$

The SLS design method demonstrated above involves implicitly the fatigue consideration for the stay cable system.

The explicit fatigue limit state design requires the stay cable system to be tested for two million cycles for specific upper stress limits and specific amplitudes. For cable stayed bridges, the upper stress limit is taken usually as 45% f_{UTS} . The amplitude is commonly taken as 160 MPa for strand systems and 200 MPa for wire systems. Upon carrying out the fatigue test, appropriate safety factors is taken into account to specify the allowable stress variation.

R. Walter [Walter et al. 1999] suggests simplified method for the definition of the permissible stress variation $\Delta\sigma_{per}$ by establishing the Woehler curve (Figure. A3-4) from the available test results then deducing the allowable stress variation $\Delta\sigma_{per}$ by using the following equation:

$$\Delta\sigma_{per} = \frac{\Delta\sigma_{test}}{\gamma_{Mf}} \approx \frac{\Delta\sigma_{test}}{1.5}$$

Where:

Ymf = 1.5 partial safety factor (as per SETRA)

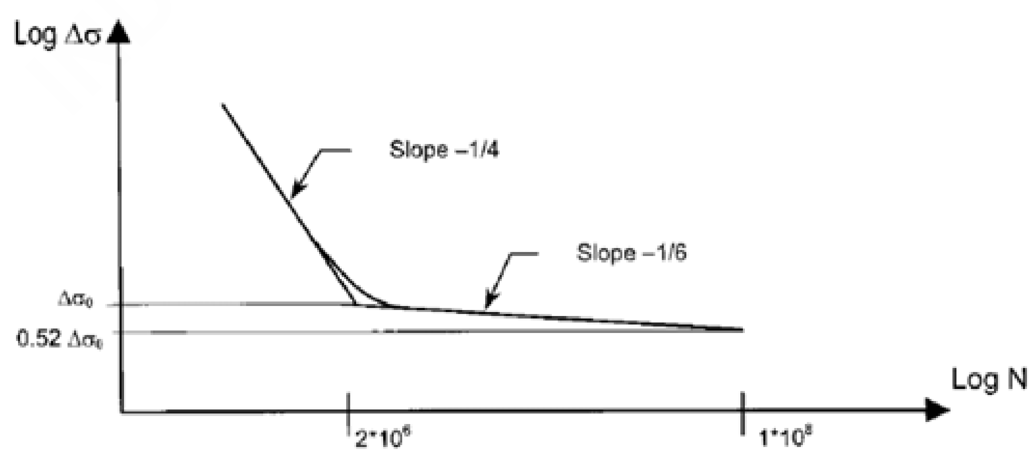


Figure A3-4: Typical fatigue Curve for Stay Cable as per SETRA 2002

Upon the determination of $\Delta\sigma_{test}$, the stress variation due to the fatigue load should be calculated (using a structural computer model) and should be less than ($0.52\ \Delta\sigma_0 /1.5$). For a stay cable system with $\Delta\sigma_0$ = 200MPa for example, the allowable stress variation due to the fatigue load = (0.52 x 200/1.5) ≈ 70MPa. The fatigue load for road bridges for design shall be as per the Eurocode EN 1991-2-2003. The stress due to fatigue live load should be less than 70Mpa as per SETRA.

Based on the previous experience with the design of cable-stayed bridges with concrete decks, the stress variations due to fatigue loads like that of the Model 3 of Eurocode EN 1991-2-2003, are often far below 70MPa. The SLS design governs therefore the required size of the stay cables. Similarly, for extradosed bridges, where the stress variations due to live loads are less than the corresponding stress variations for the cable-stayed bridges, the SLS design is expected to also govern the size of the stay cables and the permissible stress at SLS is increased as allowed for by SETRA recommendations (2002).

III. ***PTI Recommendations for Stay cable design, Testing and Installation***

The philosophy adopted by PTI for performance of the cable, is to start with the single 7-wire element fatigue capacity of 33 ksi (229Mpa) (Figure A3-5), then reduce this by a value, D_1, to cover quality assurance considerations, and a value, D_2, to cover length effects, anchorage stress risers and factor of safety. The quality assurance value considers the difference from the performance of an individual strand element compared to a group of strands forming a cable. It includes interaction effects, cable. twist, and assembly variances, and is based on experience from load testing results of many assembled cables. For strand cables the D_2 correction used by PTI is 10 ksi, (69Mpa) and the D_1 correction is 4.93 ksi. (35Mpa) Figure A3-5 shows application of these corrections, resulting in an allowable fatigue stress range for a cable comprised of multiple 7-wire strands of 19 ksi (124Mpa) at 2 million cycles, based on a maximum stress of 45% of the ultimate strength of the strand

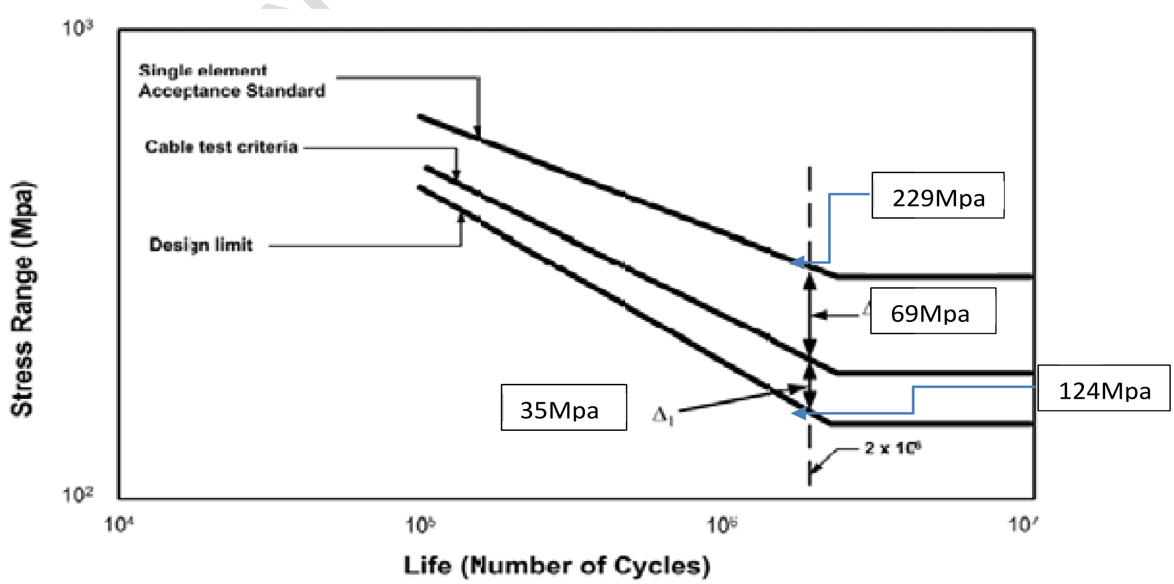


Figure A3-5: Strength resistance Factor, (PTI DC45.1-18)

The PTI 3rd edition and earlier were based on an allowable stress design procedure where the cables were sized based on a maximum stress of 0.45 f's for Group I loading, then a check of the fatigue range of 18.1 ksi based on the selected cable size. If based on this check the fatigue limit is not met, then fatigue governs, and the cable size is increased to reduce the stress range.

The current PTI specification is based on a Load Resistance Factor Design (LRFD) philosophy, meaning that the four basic limit states of service, strength, fatigue and extreme event must satisfy for the following expression:

$$\eta \sum \gamma_1 Q_i \leq \phi R_n = R_r$$

where :

Y_1 is the load factor

ϕ is the resistance factor

Q_1 is the force effect, axial load, or axial plus bending, as applicable

R_n is the nominal resistance.

R_r is the factored resistance; and

$\eta >$ 0.95 is a factor relating to ductility, redundancy, and operational importance. (Refer to Sections 1.3.3, 1.3.4, and 1.3.5 of AASHTO LRFD.). The factor η = 1 for these Recommendations.

For the limit states referred to in Table 3.4.1-1 of AASHTO LRFD, the following resistance factors shall apply:

Strength A – Axial Only ϕ = 0.65 to 0.75 per diagram Fig. A3-6

Strength B – Axial and Bending combined ϕ = 0.78

Extreme Event ϕ = 0.95

Fatigue ϕ = 1.0

In PTI recommendations", 5th Edition an attempt to include term extradosed bridges in the specification was made. These provisions allowed design to an equivalent allowable stay stress of 0.6 f's for the very specific class of bridges where the fatigue stress range is less than 2.5%. While this went part way in recognizing extradosed bridges, it allowed no flexibility on intermediate conditions where the fatigue stress is more than 2.5% but less than a typical cable-stayed bridge.

PTI, 6th Ed. edition removed the specific references to extradosed bridges in the 5th edition, to include provisions that address bridges with relatively low live load demands on the stay cables in a more general manner. This is accomplished by allowing a transition in the stay capacity for low demand cables, expressing this as a function of the total live load plus wind stress to maximum ultimate tensile strength ratio (Total LL+W/MUTS) for the cable. The material factor are considered to recognize the lower live load fatigue demand on bridges that have relatively small live load demand on the cables. For the total LL+W/MUTS ratio over 7.5% the material factor is 0.65 as for a normal cable stayed bridge. For a Total LL+W/MUTS ratio of 2.5% a factor of 0.75 is allowed, essentially allowing a maximum stay stress of 0.6 f's. A linear transition is permitted between these two limits (Figure A3-6).

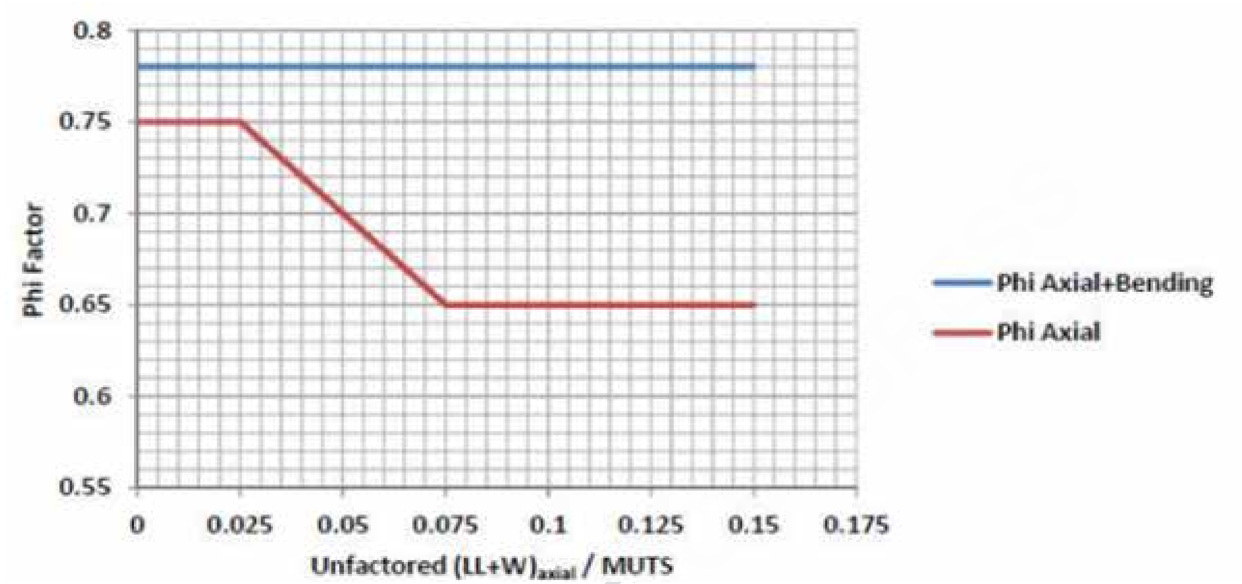


Figure A3-6: Strength resistance Factor, (PTI DC45.1-18)

The PTI Recommendations requires that the cable forces comply with a Strength Limit State and a Fatigue Limit State. For a theoretically infinite fatigue life, the maximum stress range in each cable must be less than the constant amplitude fatigue threshold stress. The fatigue load consists of a single design truck, in a single lane, and the load effect is then increased by a Dynamic Load Allowance of 15% and by a factor of 1.4 to account for longer spans of cable-stayed bridges than those for which the fatigue provisions of the AASHTO LRFD (AASHTO 2004) were originally designed. The factored load effect $\gamma(\Delta F)$ must be less than half the constant amplitude fatigue threshold $(\Delta F)_{TH}$ as described by the following equations.

$$\gamma(\Delta F) \leq (\Delta F)_{\eta}$$
$$\gamma(\Delta F) \leq \frac{1}{2}(\Delta F)_{TH}$$

where (ΔF) is the stress range due to the passage of the fatigue load

$(\Delta F)_{TH}$ is the constant amplitude fatigue threshold (taken as 110 MPa for parallel strands)

γ is the load factor of 0.75

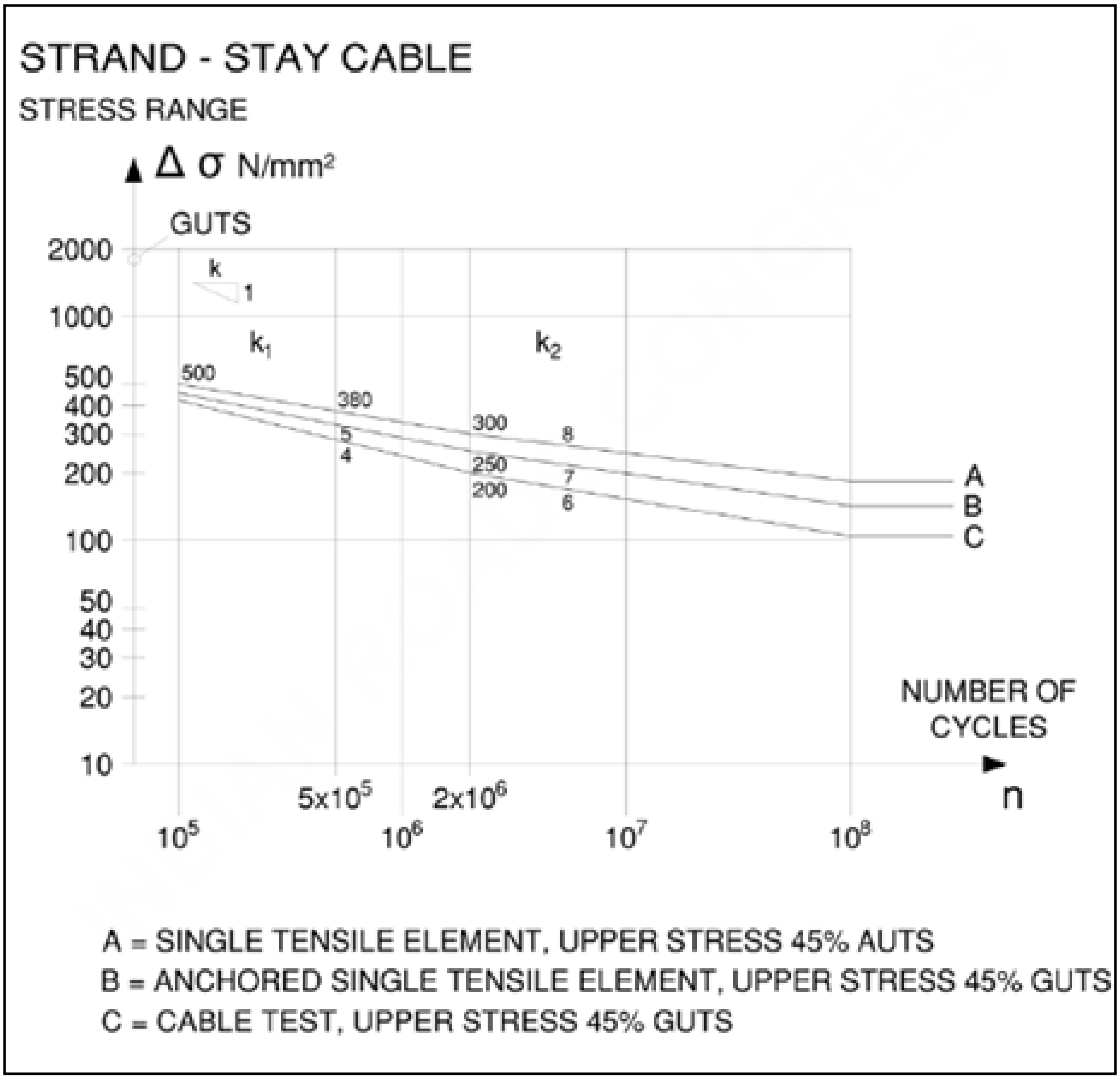
$$\therefore 0.75\ (\Delta F) \leq \frac{1}{2}(110)$$
$$(\Delta F) \leq 73\ \text{MPa}$$

For AASHTO LRFD, stress range due to fatigue live load must be less than 73Mpa.

IV. ***Acceptance of Cable systems using prestressing steel – FIB 89***

As per FIB 89, fatigue design should be carried out for stay cables by the method similar to SETRA as explained above with material factor of 1.25 and Wohler Curve C (Fig. A3-7) which represents minimum test performance requirements of stayed / Extradosed cables.

Also the permissible stresses in the stay cables system tested in accordance with FIB 89 chapter 6 shall be 50% GUTS for cable stay and for extradosed cable system shall be 60% GUTS.



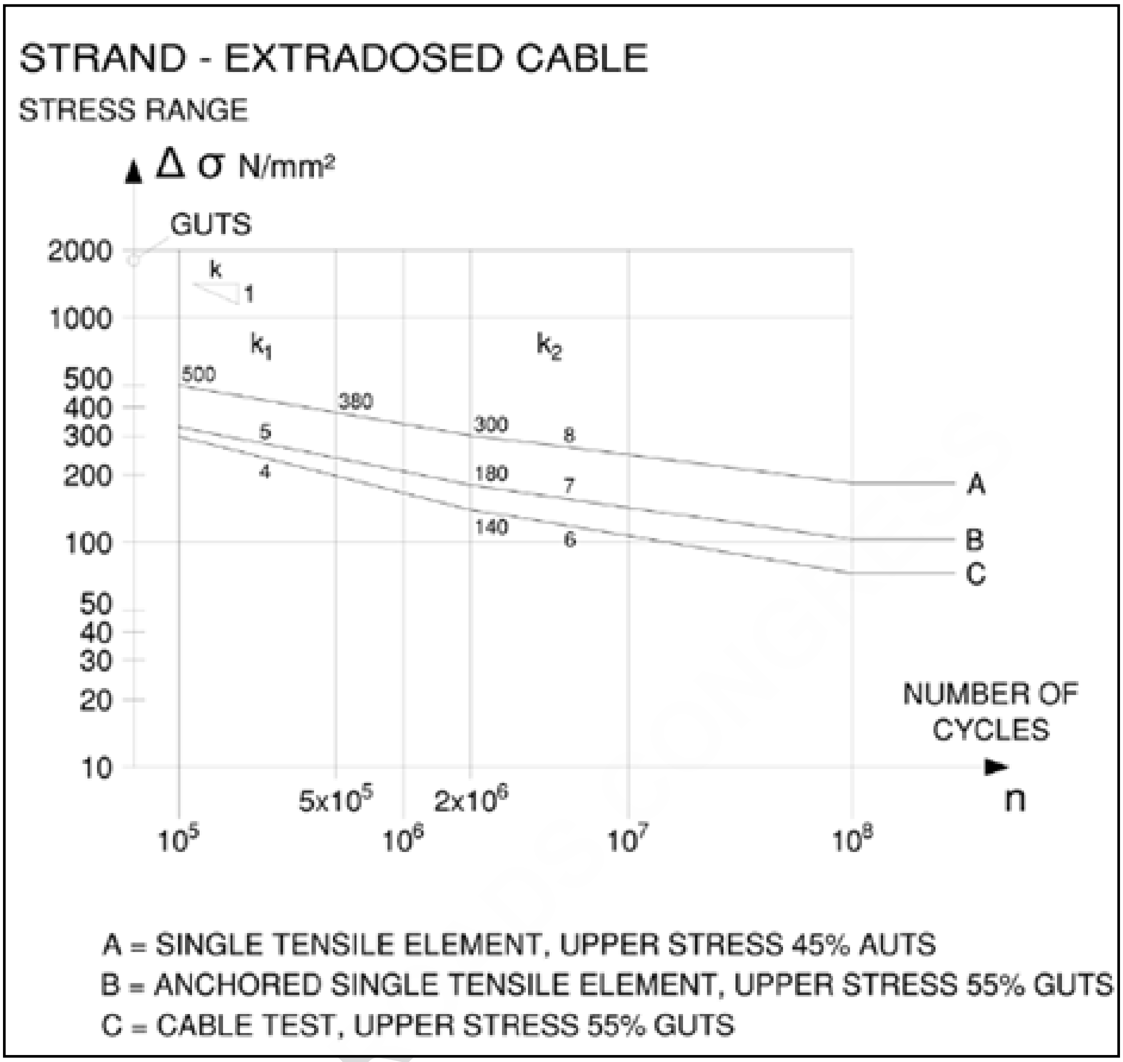


Figure A3-7: S-N Curve for cable system as per FIB 89

Annexure - A4

Dynamic Characteristics and Wind Effects of Extradosed Bridges

A4-1. Dynamic Characteristics of Extradosed Bridges

The vibration of extradosed bridges may involve a single member (deck, pylon or cable) or the entire structure. From the dynamic characteristics of some of the existing EDB's it is seen that, generally free vibration characteristics of extradosed bridges are more like that of girder bridges and are less prone to wind induced oscillations. Most of the EDB's without seismic isolation devices and with main span of about 150m, generally have a natural frequency greater than 1.0 Hz. Only few modes of vibrations (deck or the pylon modes) are lower than 1.0Hz for bridges with main span in the range of 150 to 200m. However, it is seen that EDB's with friction pendulum bearings used to reduce seismic demand by extending fundamental time period are having frequency of vibration less than 1.0 Hz. As most of the extradosed bridges are provided with PSC deck with single/multi cell concrete box girders with high torsional rigidity, the first torsional mode of vibration of deck is often found to be more than 1.0Hz.

Cables in extradosed bridges are shorter than in cable stayed bridges and are subjected to higher stresses and are less sensitive to vibrations. Periods of vibration being short, these cables are typically less sensitive to the wind induced oscillations. But other vibrations initiated by live loads can occur, generating a significant stress range with fatigue effects. In such cases, use of dampers is often required to mitigate the vibration of cables. There are many types of dampers for stay cables, for example, viscous dampers, friction dampers and oil dampers are available to mitigate cable vibrations.

A4-2. Desktop Study to check the susceptibility of EDB's to dynamic wind effects

The wind effects on extradosed bridges (depending on span arrangement) consist of static and dynamic wind effects. The methodology to be adopted to assess the susceptibility of various structural components of EDB is given below

Step 1: Structural and Wind Characteristics

The input required for the desktop study are (i) Main span length of the bridge (L) (ii) Deck width (B) and (ii) Deck depth(d) at mid span and at the location of pylon. In general, for extra-dosed bridges, the girder height is usually of the order of L/35~L/45 at the pylon and L/50~L/60 at midspan, where L is the main span length.

For assessing the wind characteristics at the location of the bridge the following parameters are considered. (i) Level of bridge deck above the mean ground level, (ii) Basic wind speed zone depending on the location of bridge as per IRC:6, (iii) Terrain category at the location of bridge, (iv)Turbulent intensity (I_u) at the level of bridge deck, which depends on the terrain category, and can be computed on that basis. Also, compute the design wind speed (V_d based on 3sec gust) at the location of bridge, taking into account the effects of terrain, topography and cyclonic conditions at the bridge deck level. For design wind speed and turbulence intensity computation IS: 875:Part3:2015 shall be referred till wind load clause of IRC:6 is revised.

Step 2: Checking the susceptibility for dynamic wind action as given in Table A4-1 for bridge deck

Table A4-1: Checking the susceptibility of Bridge Deck of EDB to dynamic wind action

Type of Wind induced oscillations of deck	Susceptibility Parameter	Limiting value	Inference	Further Action Required
Single degree of flutter in Torsion	$L\ V_d/B$	$L\ V_d/B > 780$	Deck is susceptible to torsional flutter	See note (i) to (iii) below.
Galloping	LV_d/B and B/d	For composite decks $LV_d/B > 475$ and $B/d < 5$, then susceptible to galloping	Bridge deck is susceptible to galloping	See note (i) and (iv)
Vortex induced oscillation	LV_d/B and Iu	$LV_d/B > 275$ and $I_u < 0.2$	Deck is susceptible to vortex induced oscillation	See Note (i) and (v) below

Notes

(i). If the EDB's to susceptible to wind action as shown in Table A4-1, a free vibration analysis is carried out after 3D- modelling of the bridge with appropriate boundary conditions.

(ii). Identify the respective vibration modes of deck, tower and cables (by examining the mode shapes of vibration) with frequency values less than 1.0 Hz, which are susceptible to wind induced oscillations. Check the ratio of first torsional to vertical mode of bridge deck and if this ratio is more than 2, the possibility of occurrence of classical flutter is ruled out.

(iii). Compute the critical wind speed for initiation of flutter (V_{crf}). When the width-to-depth ratio of the bridge deck section B/d=4~8, the flutter critical wind speed of bridge with main span less than 300m may be calculated according to the relation: $V_{crf} = 5f_t \cdot B$, where f_t is the first torsional frequency of bridge deck in Hz. If the estimated critical wind speed for flutter is greater than 1.65 times design wind speed (V_d) at bridge location (3sec), then occurrence of torsional flutter could be ruled out.

(iv). Estimate static coefficients for bridge deck from CFD studies or wind tunnel tests on sectional model at different angle of attacks. The susceptibility of bridge deck to galloping shall be evaluated by checking signs of the time-averaged static wind coefficients for lift and moment at zero angle of attack. A negative slope of static lift (or moment) coefficient usually indicates a tendency for galloping. Also it can be checked using Den Hertog Criterion i.e., at wind angle α =0, if (C_d +($dC_L/d\alpha$) <0

where: α is the angle of attack in degrees, C_L is the aerodynamic lift coefficient and C_D is the aerodynamic drag coefficient of bridge deck at a wind angle zero.

(v). Identify the vertical modes of bridge deck with frequency less than 1.0Hz by inspecting the various mode shapes of vibration

(a) Compute the respective critical wind speed for vortex excitation for vertical modes of bridge deck with frequency less than 1.0 Hz, $V_{cr}= f_v d/S$, where f_v is the vertical frequency of bridge deck, 'S' Strouhal Number for box -girder bridge deck, which is in the range 0.1 to 0.15. If the computed V_{cr} values are lower than design wind speed at the deck level, then these modes are prone to vortex excitation. Once vortex-induced oscillations have been established at a given wind speed, the vortex shedding frequency may "lock" into the bridge natural frequency as long as the wind speed does not vary significantly. Therefore, vortex shedding often persists over a narrow range of wind speeds. It may be noted that vortex shedding is most noticeable in smooth flows and tends to decrease as the turbulence of the wind increases.

The effect of vortex resonance may not be considered when the fundamental frequency of structural component is larger than 5Hz.

(b) Compute the amplitude of vortex excitation for deck modes which are susceptible to vortex excitation.

The amplitude of vortex induced oscillation of box girder deck could be estimated using the following:

The amplitude of vibration of deck due to vortex excitation needs to be lower than allowable amplitude of oscillation determines from bridge user's comfort criteria. (Japanese/Chinese specification).

The vortex induced oscillatory amplitude in vertical direction of a box girder bridge deck may be computed according to the following equations:

$$h_a = \frac{E_h E_{th}\, B}{2\pi m_r \zeta_s} \tag{A4.1}$$

where

$$m_r = \frac{m}{\rho B^2} \tag{A4.2}$$

$$E_h = 0.065\,\beta_{ds}\,\frac{d}{B} \qquad \text{A4.3}$$

$$E_{th} = 1 - 1.5\,\beta_t\,\sqrt{\frac{B}{d}}\;I_u^2 \geq 0 \qquad \text{A4.4}$$

$$I_u = \frac{1}{ln\frac{Z}{z_0}} \qquad \text{A4.5}$$

h_a—The vertical vortex induced oscillatory amplitude (m);

m—The linear mass of bridge (kg/m). It may be taken as average value at 1/4 span as for the bridge with variable cross-section along with half of the mass of extradosed cable system

ζ_s— critical damping ratio of bridge deck

β_{ds}—The shape correction coefficient, which shall be taken as 1 for a bluff -body with width larger than ¼ of effective height.

B—The width of deck (m)

d—The depth of deck (m)

β_t—The coefficient shall be taken as 1

I_u—The turbulence intensity;

Z—The reference altitude of deck (m)

z_0—The terrain roughness height at bridge site (m) for different terrain categories (TC1 to TC 4) which may be selected according to IS: 875: Part 3: 2015 till it is included in IRC: 6

(c) Check whether amplitude of oscillation due to vortex excitation (h_a) of bridge deck is less than $0.04/f_v$ otherwise it may cause discomfort to users [5, 6].

(d) If yes, then, consider improving geometry of deck or by installing a TMD

Step 3: Susceptibility of Pylon to Dynamic Wind Action

(v) Pylons of EDB's may be prone to dynamic wind action, i.e., vortex induced oscillation or buffeting, if the frequency of lateral or longitudinal mode of towers from note (i) above is less than 1.0 Hz or When ratio of height to minimum lateral dimension of tower is more than 5.0. The transverse or lateral and longitudinal modes of vibration of pylon are identified by examining the mode shapes of vibration obtained from the free vibration analysis.

For estimation of vortex induced oscillation of upper pylon, Strouhal number (S) of 0.12 may be used

$V_{cr}=f_p d_p / S$ A.4.6

If the height of free-standing pylon (i.e., during construction stage) is more than 20m, then safety against dynamic wind shall be considered.

Step 4: Check for Wind induced cable vibration in Extradosed Bridges

The wind-induced vibration of extradosed cables can be categorized as vortex induced oscillation, rain-wind induced vibration and wake galloping. The cable vibrations are likely to cause the fatigue due to secondary bending at the region near cable to deck /pylon anchorage, but also cause discomfort to bridge users.

Therefore, it is necessary to check the susceptibility of vibration of extradosed cable and incorporate countermeasures to mitigate the dynamic wind effects.

Dynamic effects of the wind especially in cables with length greater than 80m shall be considered as follows.

a) Vortex-induced Vibration (VIV)

VIV is due to the alternate shedding of vortices from opposite sides of the strand, inducing a periodic force in the cable. As steady wind is required for this effect to occur, the most damaging vibrations generally occur at low wind velocities. Vortex shedding is expected to occur when its frequency becomes approximately the same as the natural frequency of a cable, that is to say, a wind velocity approaching the resonance inducing velocity. In order to avoid this phenomenon, the natural frequency of cables should not be the same as the vortex-shedding frequency described below:

$V_{cr}= f_c d_c/S$ A4.7

Where

f_c- frequency of vibration of the cable obtained from free vibration analysis

V_{cr} – Critical wind speed for vortex excitation of extradosed cable (m/s)

S - Strouhal number 0.2 for circular cable

d_c - outer diameter of the cable (m)

Also, the Scruton number (Sc) for the extradosed cable shall be computed using Eq. A.4.8. The Scruton number for stay cables without any dampers is typically Sc less than 1, However, if S_c more than 2.5 I, then vortex induced oscillation is restricted.

VIV can be mitigated by reducing or eliminating the underlying excitation along the lengths of the extradosed cables that drives cable oscillation by introducing projections, or texturing, of the cable pipe. This roughens the surface of the cable and presents an irregular surface to the wind flow.

b) Rain-wind induced vibration.

Natural frequency of cables shall be obtained from three-dimensional free vibration analysis of the EDB system. Rain-wind induced vibration occurs in cables with smooth surface protective tube and with natural frequency in the range of 1 to 3 Hz. The occurrence wind velocity range is around 6 to 18 m/s. The wind must be from an oblique direction, between 30^o and 80^o from the perpendicular to the cable, so that the wind will tend to lift the cable. Below the stated wind speed range, the top rivulet does not form because the wind is insufficient to prevent it from running down the side of the cable. Above the range, the wind forces tend to blow the rivulets off the cable. Wind turbulence also prevents the rivulets from forming.

In order to avoid the onset of rain–wind oscillation it has been suggested that the Scruton number (S_C) for the cable should be at least 10 according to the following formula specified in the [4]

$$S_C = \frac{m\xi}{\rho {d_c}^2} \geq 10 \quad \text{A.4.8}$$

where

m _ cable mass per unit length (kg/m)

ξ- critical damping ratio $\xi = \delta / 2\pi$, and δ is logarithmic decrement of cable. For preliminary estimation, the critical damping ratio may be taken as 0.1% (for cables without dampers) for all type of cables. Alternatively, the critical damping ratio values certified by the cable manufacturer shall be used.

ρ is the air density (kg/m^3)

d_c is the cable diameter (m)

Rain–wind induced oscillation is therefore unlikely to occur if the Scruton number satisfies Eq. A.4.8

Where extradosed cable pipes have effective surface texturing (special helix or dimpled stay pipes) the above requirement may be relaxed such that $S_C \geq 5$.

c) Wake galloping

When two cables are aligned parallel and close to each other, the downstream cable can be forced into unstable oscillations which are called wake galloping. Occurrence of the wake galloping depends on spacing between the cables. Wake galloping is reported to be observed when spacing between the cables is about 2.5 times of cable diameter. However, it is found that the condition does not occur for cable spacing larger than 5 d_c, where d_c is the diameter of the cable.

A4-3. **Design Wind load**

Wind load on respective components shall be assumed to be distributed on the area exposed to the wind. The wind load on a bridge shall be calculated for (i) Bridge deck between expansion joint to expansion joint (with and without live load at service and construction stages) (ii) cable system (iii) pylon

(a) Deck

To compute the design wind pressure acting on various components of an extradosed bridge during construction stage and operational stage, IRC: 6 shall be referred. For single box girder decks the drag coefficient may be taken for construction stage and operational stage or depending on the B/d ratio from IRC :6. Aerodynamic force coefficients obtained from wind tunnel testing for one of the existing extradosed bridge are given in Appendix I.

(b) Cables

The drag force from the wind per unit length of the cable shall be calculated using following equations

$$F_D = \frac{1}{2}\,\rho\, V_Z\, d_c\, C_D G \qquad \text{A.4.9}$$

where,

ρ- is the density of air (1.20 kg/m^3 for standard temperature and pressure)

V_z - Hourly mean speed of wind in m/s at mean height H of cable as per IRC 6

G - Gust factor depending on the terrain category.

$C_{D\text{ - Drag}}$ coefficient for the cable shall be taken as 1.2 if $d_cV_{z<}$ 6m^2/s and 0.8 if $d_cV_{z>}$ 6m^2/s.

(c) Pylon:

The wind load acting on pylon shall be determined for a wind speed at a height 0.65 times height of pylon from mean ground/water surface.

The pylon may be treated by the quasi-static approach for wind load computation, provided that, the following criteria is satisfied:

a) ratio of height to minimum lateral dimension is less than about 5.0 and

b) natural frequency in the first mode of pylon obtained from free vibration analysis is more than 1.0 Hz.

For the computation of along wind forces, the drag coefficient for pylon with square cross-section shall be taken as 2.0 or depending on the H/d ratio and t/b ratio as given in IRC: 6.

If the conditions stipulated in a) and b) are not satisfied, the pylon may be prone to buffeting, and Gust factor method as given in IS: 875: Part 3:2015 [4] shall be used for computation of along wind forces after computing gust factor, considering both background and resonance component and it shall be used in conjunction with hourly mean wind pressure.

A4-4. Numerical Simulation studies using Computational Fluid Dynamics

Numerical simulation based on computational fluid dynamics (CFD) provides an alternative to as wind tunnel testing, which often proves to be expensive and time-consuming. Also, CFD enables simultaneous determination of force coefficients, pressure distributions, structural response, and flow visualization of structures in various flow fields. This feature in the application of CFD is often helpful and essential for the understanding of the fluid-structure interaction mechanisms that give rise to wind induced vibration.

A4-5. Wind Tunnel Studies

Wind tunnel studies on sectional model shall be carried out for EDB's if the criteria given in Table A4-1 are not satisfied of for EDB with main span more than 250m length.

(i) Sectional model tests shall be carried out on as large a scaled model as possible. Sectional model tests shall be performed in both smooth and turbulent flows, covering wind speeds from the lowest practical range, up to and beyond the project specific stability criteria.

(ii) Steady-state force and moment coefficients should be measured outside the vortex shedding lock-in range at speed lower than the flutter speed.

(iii) Static force and moment coefficients shall be measured for wind angles ranging, as a minimum, between ±6° in increments of 2°.

(iv) Drag coefficients shall be normalized with respect to depth of deck (including crash barrier and railings)

Whereas lift and moment coefficients should be normalized with respect to width (edge to edge) of deck cross-section.

(v) Structural damping ratios shall be set in the range of their expected full-scale values when dynamic sectional model is used for checking the aerodynamic susceptibility.

REFERENCES:

1. Hunyadi, M (2009).” Flutter Analysis of an Extradosed Bridge in Hungary”, EACWE 5, Italy, 19-23, July 2009)

2. Specifications for Design and Construction of Cable-Stayed Bridge and Extradosed Bridge, Japan Prestressed Concrete Institute, 2012.

3. Fujino and Yoshida (2002) “Wind-Induced Vibration and Control of Trans-Tokyo Bay Crossing Bridge. Journal of Structural Engineering, Vol. 128, No. 8.

4. IS:875 :Part 3:2015 -Design Loads (other than Earthquake) for Buildings and Structures- Code of Practice Part 3 Wind Loads (Third Revision) , BIS, New Delhi

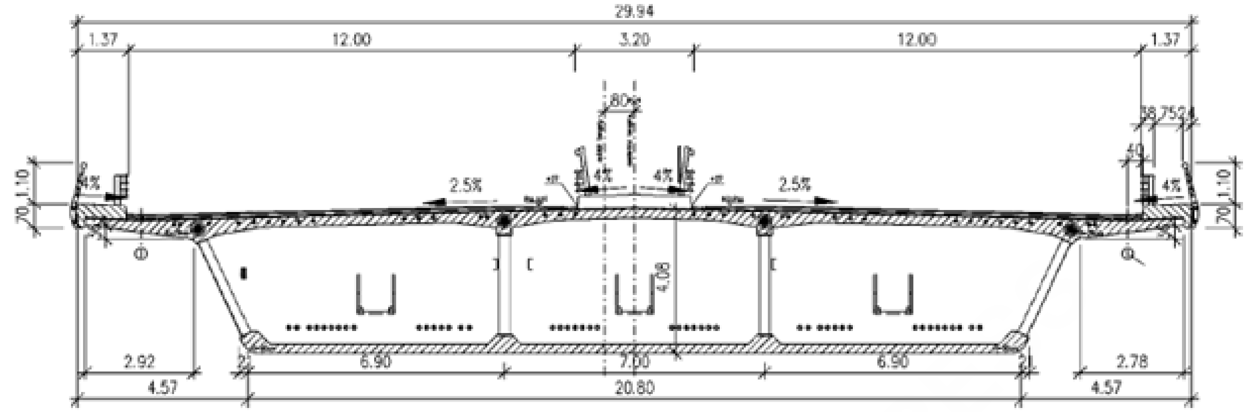
5. JTG/T D60-01-2004: Wind Resistant Design Specifications for Highway Bridges” Peoples Republic of China

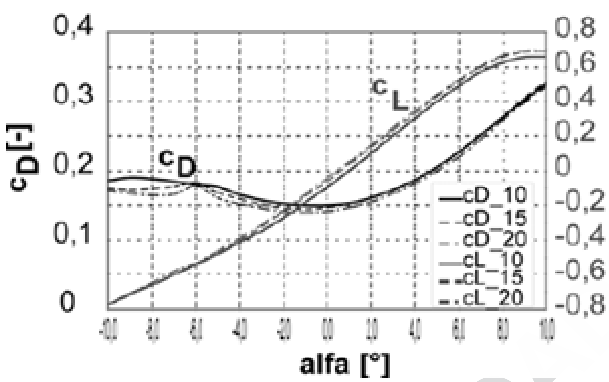
6. H. Sato and Ken-Ichi-Ogi “ Study on Performance based Wind Resistant Methods for Wind Effects on Highway Bridges

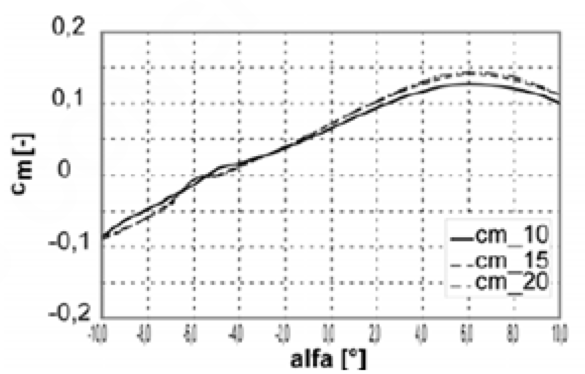
7. Design Guidelines for Arch and Cable Supported Signature Bridge, Publication No. FHWA-NHI-11-023, 2012

Appendix - 1

Aerodynamic force coefficients for an Extradosed Bridge in Hungary obtained through wind tunnel studies [1]







(Note: In the above Fig C_d has been normalized wrt to deck width B. To obtain the C_d normalized with respect to deck depth, the C_d value needs to be multiplied by B/d)

Annexure - A5

(Pertaining to Chapter 8)

A5-1. APPROACHES OF INTERNATIONAL GUIDELINES FOR TESTING OF STAY CABLE

Table A5-1

INTERNATIONAL CODES	SETRA [1]	fib-89 (CEB-FIP) [2]	PTI (7^{th} EDITION) [3]
1 CABLE STAY BRIDGES	✓	✓	✓
2 EXTRADOSED BRIDGES	✓	✓	
UPDATED	First print published in 2002 & the same will not be updated further.	Latest Version published in March- 2019	Latest Version published in Nov- 2018

Notes:-

1 If for a particular project, all the cables have $D\sigma_{sls}$ (stress range in cables due to vehicular live load under frequent live load in MPa) less than 50MPa than, qualification testing (initial as well as suitability testing) requirements of the extradosed bridges shall be adopted else all the testing requirements of the cable stayed bridges shall be adopted.

2 Apart from initial & suitability testing, other material checks on strands, PE sheathed duct, filler material and HDPE duct, wedges and anchorages parts shall be also carried out [2].

A5-2. COMPARISON OF INTERNATIONAL CODES FOR STAY CABLE FOR CABLE STAYED BRIDGES & RECOMMENDED CODE TO BE FOLLOWED FOR TEST

TESTING OF CABLE STAY- ANCHORAGE ASSEMBLY ✓

OR

TESTING OF CABLE STAY- SADDLE ASSEMBLY ✓

Table A5-2

	Test Detail	SETRA [1]	fib-89 (CEB-FIP) [2]	PTI (7th EDITION) [3]	Recommended Code For Testing
S. No	**INITIAL TESTING**				
1	ANCHORAGE /SADDLE AXIAL FATIGUE & TENSILE TESTING	✓✓	✓✓	✓✓	**fib-89**
2	ANCHORAGE /SADDLE BENDING FATIGUE & TENSILE TESTING		✓✓		**fib-89**
3	ANCHORAGE / SADDLE LEAK TIGHTNESS TESTING	✓✓	✓✓	✓✓	**fib-89**
4	SADDLE FRICTION TESTING	✓	✓	✓	**fib-89**
5	FIRE RESISTANCE QUALIFICATION TESTING (OPTIONAL-SPECIFIC TO PROJECT REQUIREMENT)- Given in PTI only			✓	**PTI**
	SUITABILITY TESTING				
1	SINGLE ELEMENTS (STRAND ALONE)-FATIGUE TESTING	✓	✓	✓	**fib-89**
2	ANCHORAGE AXIAL FATIGUE & TENSILE TESTING	✓✓	✓✓	✓✓	**fib-89**
3	ANCHORAGE BENDING FATIGUE & TENSILE TESTING	NOT REQUIRED TO BE PERFORMED			
4	ANCHORAGE LEAK TIGHTNESS TESTING	NOT REQUIRED TO BE PERFORMED			
5	SADDLE FRICTION TESTING	✓	✓	✓	**fib-89**
6	FIRE RESISTANCE QUALIFICATION TESTING (OPTIONAL-SPECIFIC TO PROJECT REQUIREMENT)- Given in PTI only			✓	**PTI**

A5.3 COMPARISON OF INTERNATIONAL CODES FOR STAY CABLE TESTING FOR EXTRADOSED BRIDGES & RECOMMENDED CODE TO BE FOLLOWED FOR TEST

TESTING OF EXTRADOSED STAY- ANCHORAGE ASSEMBLY ✓

OR

TESTING OF EXTRADOSED STAY- SADDLE ASSEMBLY ✓

Table A5-3

	Test Detail	SETRA [1]	fib-89 (CEB-FIP) [2]	PTI (7^{th} EDITION) [3]	Recommended Code For Testing
S. No	**INITIAL TESTING**				
1	ANCHORAGE /SADDLE AXIAL FATIGUE & TENSILE TESTING	✓✓	✓✓		**fib-89**
2	ANCHORAGE / SADDLE LEAK TIGHTNESS TESTING	✓✓	✓✓		**fib-89**
3	SADDLE FRICTION TESTING	✓	✓		**fib-89**
4	FIRE RESISTANCE QUALIFICATION TESTING (OPTIONAL-SPECIFIC TO PROJECT REQUIREMENT)- Given in PTI only			✓	**PTI**
	SUITABILITY TESTING				
1	SINGLE ELEMENTS (STRAND ALONE)-FATIGUE TESTING	✓	✓		**fib-89**
2	ANCHORAGE AXIAL FATIGUE & TENSILE TESTING	✓✓	✓✓		**fib-89**
3	ANCHORAGE LEAK TIGHTNESS TESTING	NOT REQUIRED TO BE PERFORMED			
4	SADDLE FRICTION TESTING	✓	✓		**fib-89**
5	FIRE RESISTANCE QUALIFICATION TESTING (OPTIONAL-SPECIFIC TO PROJECT REQUIREMENT)- Given in PTI only			✓	**PTI**

A5.4. Applicable Property, Acceptance Criteria & Test Method & Frequency of Testing of Materials

Table A5.4.1: Recommended Values for Prestressing Steels for Cables

Type of steel	E-modulus	Diameter	Tensile strength	Cross section	Recommended tolerances on mass	Characteristic ultimate force
	GPa	mm	MPa	mm^2	%	kN
Strands	195 ± 5%	15.2	1860	140	± 2	260.4
		15.7	1770	150	± 2	265.5
			1860			279.0
Wires	205 ± 5%	6	1770	28.27	± 2	50.1
		7	1670	38.5		64.3
		7	1770	38.5	± 2	68.1

Table A5.4.2: Mechanical Characteristics for Prestressing Steels

Property			Acceptance Criteria	Test Method
Elongation at maximum force			Not less than 3.5%	ISO 15630-3[S7]
Constriction at break			Ductile break visible to naked eye, constriction coefficient ≥25%	ISO 15630-3
Relaxation 1000h at 0.70 AUTS, 20^{0}C			Not more than 2.5%[1)]	ISO 15630-3
Fatigue stress range with an upper stress limit of 0.45 AUTS	Strands	300 MPa	At least $2x10^6$ cycles	ISO 15630-3
	Wires	370 MPa		
Fatigue stress range with an upper stress limit of 0.45 AUTS (alternative to 2 x 10^6 cycles)	Strands	380 MPa	At least 5 $x10^5$ cycles	ISO 15630-3
	Wires	465 MPa		
Fatigue stress range with an upper stress limit of 0.45 AUTS (alternative to 5 x 10^5 cycles)	Strands	500 MPa	At least 1 $x10^5$ cycles	ISO 15630-3
	Wires	610 MPa		
Tensile test after the fatigue test			95% of AUTS	ISO 15630-3
Deflected tensile test of Strands			Not more than 20%	ISO 15630-3

[1)]The values are applicable for testing of samples taken just after manufacturing.

Above specified properties apply to hot dipped metallically coated or epoxy coated prestressing steel. However, they also have to be satisfied after application of the filling material and sheathing.

Table A5.4.3: Requirements for metallic coatings on prestressing steel[S12]

Property		Acceptance Criteria	Test Method
Zinc or zinc/aluminium coating	-Continuity	No defects	EN 10244-1 and 2[S13]
	-Weight of coating	190 to 350 g/m^2	
	-Adherence	No defects, cracks	
	-Appearance	Smooth, no zinc drop	

Table A5.4.4: Requirements for epoxy coatings on prestressing steel

Property	Acceptance Criteria	Test Method
Coating thickness	380 to 1140 µm	ASTM A 882/A882M[S14]
Salt spray test	No corrosion, holidays[1)] nor other coating damage present after 3,000 hours exposure to salt spray (fog)@ 70% GUTS	ASTM B117[S4]
Chloride permeability	No chloride penetration detected	FHWA-RD-74-18
Chemical resistance	2 types of samples tested: Normal epoxy coated strand and sample with 1/4" hole intentionally drilled into coating. No coating damage after 45 days immersion. Steel corrosion at drilled hole did not cause adjacent epoxy coating to soften, blister, nor lose bond with adjacent steel.	ASTM G 20[S49]
Impact test	No cracking, shattering or bond loss	ASTM G 14[S23]
Sand abrasion test	< 10 mils (0.25 mm) loss of coating	ASTM D 968[S50]
Bending test	No cracking or debonding when bending strand around mandrel 32 times nominal strand diameter	ASTM A 370[S51]
Adhesion of coating	No coating peel-off is found after the break in the tensile test except in the area near breaking position and chucking position	ASTM A 882/A882M[2)]

[1)] Holiday detection: Dry-Type continuous in line detector with minimum voltage of 3KV would be applied.

[2)] No cracks visible to the unaided eye shall occur in the coating upto an elongation of 1%, or no coating peel-off is found after the break in the tensile test except in the area near breaking position and chucking position.

Table A5.4.5 Grease specification

Property		Acceptance Criteria	Test Method
Cone penetration,60 strokes at 25^0 C (1/10 mm)		220 – 300	ISO 2137[S24] or ASTM D217[S25]
Dropping point		≥ 150^0 C	ISO 2176[S26] or ASTM D566[S27]
Heat resistance, flash point		≥ 250^0 C	EN ISO 2592[S40]
Oil separation	At 40^0C	At 72 hours : ≤ 2.5% At 7 days : ≤ 4.5%	BS 2000-121[S28] DIN 51 817[S29] NFT 60 – 191[S30] or ASTM D 6184[S31]
	At 100^0C	At 50 hours : ≤ 4%	
Oxidation stability		100 hours at 100^0C: ≤ 0.06 MPa 1000 hours at 100^0C: ≤ 0.2 MPa	DIN 51 808[S32] Or ASTM D942[S33]
Corrosion Test[1)]	168 hours at 35^0C	No corrosion	EN ISO 9227 (NSS test)[1),S34]
	168 hours at 35^0C	No corrosion	EN ISO 6270-2 (AHT test)[S35] Or ISO 9227 (distilled water instead of NSS)
Rust prevention		Grade : 0	ISO 11007[S36] , solution of sodium chloride
Content of aggressive elements	Cl^-,S^{2-}, NO_3^-	≤ 50 ppm (0.005%)	NFM 07-023[S37,2)]
	SO_4^{2-}	≤ 100 ppm (0.010%)	NFM 07-023[2)]

[1)]Test sample consists of a structural steel plate S355 with a surface roughness comparable to prestressing wire and strand. The plate is covered with a layer of grease of a maximum thickness of 125µm. This thickness can be controlled with eddy current or by measuring the mass.

[2)]Applied accordingly to grease. Requirements for grease pertaining to fluidity, de-oiling, water absorption, saponification, micro-biological resistance, coefficient of expansion must be satisfied or declared.

Table A5.4.6: Wax specification

Property	Acceptance Criteria	Test Method
Congealing point	≥ 65^0C	ISO 2207[S38]
Heat resistance, dropping point	≥ 60^0C	ISO 2176[S39]
Heat resistance, flash point	≥ 250^0C	ISO 2592[S40]
Penetration (1/10 mm) at 25^0C	≤ 125 (in 1/10 mm)	ISO 2137[S41]
Cold resistance	No cracking at -40^0C	ISO 2137[S41]
Bleeding at 40^0C	≤1.0%	BS 2000-121[S28] DIN 51 817[S29] NFT 60 – 191[S30] or ASTM D 6184[S31]
Resistance to oxidation 100 hours at 100^0C	≤ 0.03 MPa	ASTM D942[S33]

Copper-strip corrosion 100 hours at 100^{0}C		Class 1a	ISO 2160[S42]
Corrosion Test[1)]	168 hours at 35^{0}C	No corrosion	EN ISO 6270-2 (AHT test)[S35] Or ISO 9227 (distilled water instead of NSS)[S34]
	168 hours at 35^{0}C	No corrosion	ISO 11007[S36] , solution of sodium chloride
Content of aggressive elements	Cl^-,S^{2-}, NO_3^-:	≤ 50 ppm (0.005%)	NFM 07-023[S37]
	SO_4^{2-} :	≤ 100 ppm (0.010%)	NFM 07-023

[1)]Test sample consists of a structural steel plate Fe 510 with a surface roughness comparable to prestressing wire and strand. The plate is covered with a layer of wax of a maximum thickness of 125μm. This thickness can be controlled with eddy current or by measuring the mass.

Table A5.4.7: Characteristics of PE sheathing

Property	**Acceptance Criteria**	**Test Method**
Melt index	To be declared	ISO 1133[S16]
Specific weight, Density	≥ 0.94 g/cm^3	ISO 1183[S17]
Carbon black	2.3 ± 0.3%	ISO 6964[S18]
Dispersion of the carbon black	Index is max. 3	ISO 18553[S19]
Distribution of the carbon black	Index is max. C 2	ISO 18553
Tensile strength 50 mm/minute (test speed)	≥ 22 MPa on raw material ≥ 18 MPa on sheathing	ISO 527-2[S20]
Elongation at break at 23^0 C 50 mm/minute (test speed)	≥ 600% on raw material ≥ 250% on sheathing	ISO 527-2
Elongation at break at -20^0 C 50 mm/minute (test speed)	≥ 150% on raw material ≥ 100% on sheathing	ISO 527-2
Thermal stability under O_2	≥ 30 minutes at 210 ^{0}C, without degradation	ISO11357-6[S20]

Table A5.4.8: Characteristics of the sheathed cable strands

Property	**Acceptance Criteria**	**Test Method**
Sheathing thickness	≥ 1.5 mm	Calibrated gauge
Quantity of filling material	≥ ± 20% of specified value	Calibrated gauge
Friction resistance (specimen length of 0.3)	≥ 667 N	XP A 35-037.1[S5]
Leak tightness / migration[1)]	Increase mass due to absorption Of water shall be less than 0.8 g.	XP A 35-037.1 Static test
Impact resistance	No perforation of the sheathing	XP A 35-037.1, 3 ASTM G14[S23]

[1)]The values are applicable for testing of samples taken just after manufacturing.

Table A5.4.9: Characteristics of HDPE cable pipe

Property	Acceptance Criteria	Test Method
Melt index	To be declared	ISO 1133[S16]
Specific weight, Density	≥ 0.94 g/cm^3	ISO 1183[S17]
Carbon black[1)]	2.3 ± 0.3%	ISO 6964[S18]
Dispersion of the carbon black[1)]	Index is max. 3	ISO 18553[S19]
Distribution of the carbon black[1)]	Index is max. C 2w	ISO 18553
Tensile strength 50 mm/minute (test speed)	≥ 22 MPa on raw material ≥ 18 MPa on pipe	ISO 527-2[S20]
Elongation at break at 23^0 C 50 mm/minute (test speed)	≥ 600% on raw material ≥ 350% on pipe	ISO 527-2 ISO 6259-3[S46]
Elongation at break at -20^0 C 50 mm/minute (test speed)	≥ 150% on raw material ≥ 100% on pipe	ISO 527-2, ISO 6259-3
Thermal stability under O_2	≥ 30 minutes at 210 ^{0}C, without degradation	ISO 11357-6[S20]
Thermal coefficient of dilation	Value to be declared by manufacturer	ISO11359-2[S22]
Bending modulus	≥ 750 MPa at 23 ^{0}C on raw material ≥ 600 MPa at 23 ^{0}C on pipe	ISO 178[S45]
Environmental stress cracking resistance	Test with min of 600 hours Cond C F20	ISO 22088[S55] or ASTM D1693[S44]

A5.5. Quality control testing of cable systems

Property	Test Frequency
Prestressing steel according to Table A5.4.2 Mechanical characteristics for prestressing steels	
Tensile strength with elongation at maximum force	1 test per strand coil[1)]
Constriction at break	
Relaxation	1 test par batch of prestressing steel[2)]
Fatigue strength with subsequent tensile test	2 tests every 100 tonnes of strand[3)]
Deflected tensile test	1 test every 100 tonnes of strand[3)]
Metallic coating according to Table A5.4.3 Requirements for metallic coating on prestressing steel[S12]	
Continuity	1 test every 20 tonnes of strand[3)]
Weight of coating	
Adherence	
Epoxy coating according to Table A5.4.4 Requirements for epoxy coating on prestressing steel	
Coating thickness	1 test per strand coil[1)]

Salt spray test	Only for initial type testing, not applicable for factory production control
Chloride permeability	
Chemical resistance	
Impact test	
Sand abrasion test	
Bending Test	1 test per strand coil[1)]
Adhesion of coating	1 test per strand coil[1)]
Relaxation	1 test per batch of prestressing steel[2)]
Grease filling material according to Table A5.4.5 Grease specification	
Cone penetration	1 test per batch[5)] of grease
Dropping point	
Heat resistance, flash point	
Oil separation at 40° C	
Oil separation at 100° C	
Oxidation stability	Only for initial type testing, not applicable for factory production control
Corrosion test (NSS test)	
Corrosion test (AHT test)	
Rust prevention	1 test per batch[5)] of grease
Content of aggressive elements	
Wax filling material according to Table A5.4.6 Wax specification	
Congealing point	1 test per batch[5)] of wax
Heat resistance, dropping point	
Heat resistance flash point	
Penetration at 25° C	
Cold resistance	
Bleeding at 40° C	
Resistance to oxidation 100h at 100° C	Only for initial type testing, not applicable for factory production control
Copper-strip corrosion 100h at 1 00° C	
Corrosion test (NSS test)	
Corrosion test (AHT test)	
Content of aggressive elements	1 test per batch[5)] of wax
Sheathing according to Table A5.4.7 Characteristics of PE sheathing	
Melt index	1 test per batch of raw material[4)]
Specific weight, Density	
Carbon black	
Dispersion of carbon black	
Distribution of carbon black	
Tensile strength	1 test per batch of raw material[4)] 3 tests every 10 tonnes of sheathing material
Elongation at break at 23^{0} C	
Elongation at break at -20^{0} C	

Thermal stability under O^2	1 test per batch of raw material[4)]
Completed strand according to Table A5.4.8 Characteristics of the sheathed cable strands	
Sheathing thickness	1 test per strand coil[1)]
Quantity of filling material	1 test per strand coil[1)]
Friction resistance	3 tests every 100 tonnes of strand[3)]
Leak tightness	
Impact resistance	
HDPE cable pipe according to Table A5.4.9 Characteristics of HDPE cable pipe	
Melt index	1 test per batch of raw material (on raw material[4)] 1 test per size of cable pipe but at least one test per raw material batch (on pipe)
Specific weight, Density	1 test per batch of raw material[4)]
Carbon black	
Dispersion of carbon black	
Distribution of carbon black	
Tensile strength	3 tests per batch of raw material (on raw material[4)] 3 test per size of cable pipe in the project (on pipe)
Elongation at break at 23^0 C	
Elongation at break at -20^0 C	
Thermal stability under O^2	1 test per size of cable pipe but at least one test per raw material batch (on pipe)
Thermal coefficient of dilatation	Only for initial type testing, not applicable for factory production control
Bending modulus	3 tests per size of cable pipe but at least one test per raw material batch (on pipe)
Environmental Stress Cracking Resistance	Only for initial type testing, not applicable for factory production control

[1)]A coil is a delivery unit, weighing not more than 5 tonnes.

[2)]Batch corresponding to one heat number, weighing not more than 200 tonnes. Relaxation tests with minimum 120h according to ISO 15630-3 and subsequent extrapolated relaxation values to 1000h might be accepted.

[3)]Weight of bare steel

[4)]Batch corresponding to one production unit capacity, weighing not more than 30 tonnes

[5)]Quantity of product generally manufactured in one operation.

A5.6 References of Testing Standards

[S4] - ASTM B117-16 " Standard practice for operating salt spray (fog) apparatus"

[S5]- XP A35-037:2003 "Torons en acier a haute resistance proteges gaines"

[S7]-ISO 15630:2011 "Steel for reinforcement and prestressing of concrete"

[S8]- prEN 10138:2009 "Prestressing Steel"

[S13]- EN 10244:2009 " Steel wire and wire products, non-ferrous metallic coatings on steel wire"

[S14]- ASTM A 882 M-10 " Standard specification for filled Epoxy -coated Seven Wire Prestressing steel strand"

[S16]- ISO 1133:2011 " Plastics-Determination of the melt flow rate (MFR) and the melt volume rate (MVR) of thermoplastics"

[S17]- ISO 1183:2012 " Plastics-Methods for determining the density of non-cellular plastics"

[S18]- ISO 6964:2006 " Polyolefin pipes and fittings-Determination of carbon black content by calcinations and pyrolysis-Test method and base specification"

[S19]- ISO 18553:2002 " Method for the assessment of the degree of pigment or carbon black dispersion in Polyolefin pipes, fittings and compound

[S20]- ISO 527:2012 "Plastics- Determination of tensile properties"

[S22]- ISO 11359-2:1999 "Plastics- Thermomechanical analysis (TMA)-Part-2 : Determination of coefficient pf linear thermal expansion and glass transition temperature"

[S23]- ASTM G14-04 "Standard test method for impact resistance of pipeline coatings (Falling Weight Test)"

[S24]- ISO 2137:2007 "Petroleum products -Lubricating grease and petroleum -Determination of cone penetration"

[S25]- ASTM D217 -16 "Standard test methods for cone penetration of lubricating Grease"

[S26]- ISO 2176:1995 "Petroleum products -Lubricating grease -Determination of dropping point"

[S28]- BS 2000-121:2005 "Method of test for petroleum and its products. Determination of Oil Separation from lubricating grease, Pressure filtration method"

[S29]- DIN 51817:2014 "Determination of Oil Separation from lubricating grease under static conditions"

[S30]- NFT 60 191:2011 "Petroleum products and Lubricating grease- Oil Separation On storage of Lubricating Grease- Method under pressure -Static conditions"

[S31]- ASTM D6184-16 "Standard Test Method for oil separation from Lubricating grease (Conical Sieve Method)"

[S32]- DIN 51808:2015 "Testing of Lubricants; determination of oxidation stability of greases : oxygen method"

[S33]- ASTM D942-15 “Standard Test method for Oxidation Stability of Lubricating Greases by the Oxygen Pressure Vessel Method”

[S34]- ISO 9227:2017 “Corrosion tests in artificial atmospheres-Salt spray tests”

[S35]- ISO 6270-2:2005 “Paints and varnishes-Determination of resistance to humidity -Part-2 Procedure for exposing test specimens in condensation-water atmospheres”

[S36]- ISO 11007:1997 “Petroleum products and Lubricants- Determination of rust prevention characteristics of lubricating greases”

[S37]- NFM 07-023:1969 “Liquid Fuel-Determination of Chlorides in crude petroleum and petroleum products”

[S38]- ISO 2207:1980 “Petroleum waxes-Determination of Congealing point”

[S39]- ISO 2176:1995 “Petroleum products-Lubricant greases-Determination of dropping point”

[S40]- ISO 2592:2000 “Determination of Flash and fire points – Cleveland open cup method”

[S41]- ISO 2137:2007 “Petroleum products and Lubricant - Determination of Cone penetration of lubricating greases and petrolatum”

[S42]- ISO 2160:1998 “Petroleum products- Corrosiveness to copper -Copper strip test”

[S44]- ASTM D1693 “Standard test method for environmental stress cracking of ethylene plastics”

[S45]- ISO 178:2010 “Plastic, Determination of bending modulus”

[S46]- ISO 6295-3:2015 “Thermoplastic pipes-Determination of testing properties-Part 3 -Polyolefin pipes”

[S49]- ASTM G20-10 “Standard Test Method for Chemical resistance of pipeline coatings”

[S50]- ASTM D968-16 “Standard Test Methods for Abrasion Resistance of Organic Coatings”

[S51]- ASTM A370-17 “Standard Test Methods and definitions for mechanical Testing of Steel Products”

[S55]- ISO 22088:2006 “Plastics-Determination of resistance to environmental stress cracking (ECS)”

ANNEXURE A.6

ROLE OF SEISMIC ENERGY DISSIPATION, FLEXURAL PLASTIC HINGES AND THEIR POTENTIAL LOCATIONS IN DIFFERENT CONFIGURATIONS OF PYLONS/PIERS

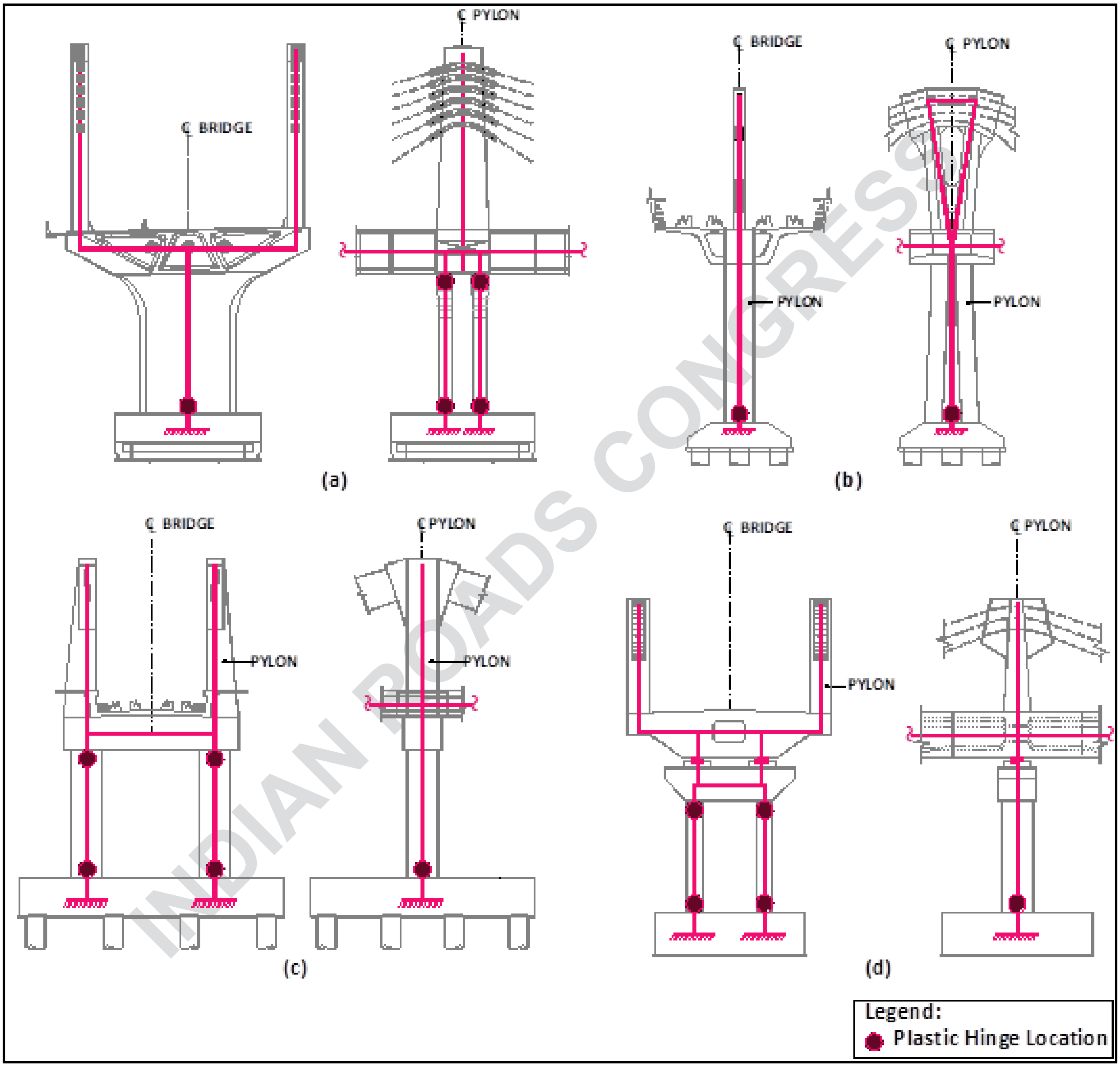
Inelastic energy dissipation plays an important role in seismic safety of structures. A structure can survive an earthquake if it has the capacity to dissipate the energy imparted by the ground motion at the same or higher rate. The capacity of a structure to dissipate energy depends on its ductility. Steel is known to be a more ductile material as compared to concrete. With proper confinement and ductile detailing, a reasonable level of ductility can be obtained in reinforced concrete in flexural action. However, it remains brittle in shear and the ductility in flexure also diminishes sharply, when it is combined with axial compression. Therefore, only those RC members which are subjected to predominantly bending type failure, with small axial loads, and in which the shear failure is avoided using capacity design, can dissipate energy during earthquakes.

In case of bridge piers and pylons, different types of configurations are used in practice. Some of these configurations involve cantilever or framed type configurations involving orthogonal members, whereas others involve triangulated systems of inclined members. The former types of pylon configuration can develop flexural hinges at the base or near the joints and can effectively dissipate energy, whereas the later types of pylons act predominantly like truss and develop large axial force in the members, which diminishes the ductility and hence the energy dissipation capacity. Accordingly, the first type of pylons can be designed using an appropriate value of response reduction factor, whereas the second type of pylons should be designed with R=1. In the first type of pylons, the potential/ desired locations of the flexural plastic hinges can be identified and ductile detailing can be performed. The ductile detailing shall consist of confinement of concrete using closely spaced stirrups of adequate size and number of legs in the plastic hinge region, and avoiding yielding at undesired locations and modes (shear) using capacity design concepts. In the second type of pylons, where the location of plastic hinges can not be identified, ductile detailing is not possible and the structure should be designed to remain elastic (Response Reduction factor, R=1.0).

The following figures provide potential locations of flexural plastic hinges in some common types of pylon configurations. The possibility of formation of flexural hinge having adequate ductility depends on the axial force in the component, which in turn depends on the inclination of the component and connection with other members. So, it is not very straightforward to identify the location of flexural plastic hinges. To identify the location of flexural plastic hinges, the demand to capacity ratio of the moment and axial force should be considered at salient locations. At the yielding of a member in bending, the axial force should be sufficiently below the axial force capacity corresponding to balanced failure.

In some cases, a member of the pylon may be capable of developing a flexural hinge in one direction of loading (longitudinal/transverse with respect to the movement of traffic) but may not be capable of flexural yielding in the other direction, because of the presence of high axial load due to large inclination in that direction. It is not very clear, how the pylons should be designed in such cases, as the earthquake is having components in both directions, and it is not clear how effective such pylons are in seismic energy dissipation. One possibility is to use different response reduction factors in the two directions.

In the following figures, the potential/desired location of flexural hinges has been indicated in both directions. The capacity at other locations shall be protected as per code IRC:SP:114.



Annexure A.7

Organisation Setup and Training Requirements for Inspection and Maintenance

This Annexure discuss about the minimum organizational setup and training requirements of personnel involved in operation and maintenance

A 7.1 Organisation Set up

For the inspection and maintenance of an extradosed bridge project a suitable organization set-up consisting of about 10 to 15 members headed by the Chief Engineer (Maintenance) or of equivalent rank in an O&M Agency outlining the responsibility of each & every staff (see Table A.7.1) shall be framed in such a manner that each element of the project shall be checked /inspected properly. The service level of each element of the project shall be assured for safe and undisturbed traffic movement.

Table A 7.1 Organisation Setup for inspection and Maintenance

Chief Engineer (Maintenance)		
Executive Engineer level officer (Maintenance) for Bridges	Executive Engineer level officer (Roads)	Executive Engineer level officer (Maintenance) for Electrical components
Maintenance Engineer (Main EDB/ Approach spans	Maintenance Engineer (Roads)	Maintenance Engineer (Electrical)
Inspector (Bridges	Inspector (Roads)	Inspector (Electrical)

The Chief Engineer (Maintenance), the Executive Engineer (Maintenance), Maintenance Engineer for Bridge and Roads should be qualified civil engineers with at least 20, 15 and 10 years of working experience respectively, major part of which should be in inspection and maintenance of long span bridge structures.

The proposed organization set-up as in Table A 7.1 may, however, be changed to suit the requirement of maintenance activities at site. The above organization set-up shall be augmented by Supervisors, System-in -Charge, Electrical, Secretary to the Chief Engineer (Maintenance), CAD draftsman, Enumerator, Computer operator and other office assistants as and when required for preparation of drawings, record keeping, estimating and help in other miscellaneous items.

It is essential that the inspection and maintenance team get familiarize themselves thoroughly with design, construction and maintenance of this type of bridges.

Responsibilities

The duties of each inspection and maintenance officers as well assistants are mentioned below.

<u>Duties of Chief Engineer (Maintenance) (one no.)</u>

i. *Organize fortnight meeting to review the maintenance activities.*

ii. *Approval of the maintenance activities*

iii. *Allotment of fund*

iv. *Monitoring the maintenance works.*

v. *Coordination with various agencies like traffic police, local administration, publicity media etc., in case of emergent repairs interruption to traffic by road blockage, etc.*

vi. *Monitoring the all-round safety*

Duties of Executive Engineer (Maintenance) (one each for bridge, road and Electrical Components if any)

i. Conducting special inspection, surveys and site investigations

ii. Finalization of inspection report

iii. Checking the cause of defects, structural inadequacy

iv. Finalization of maintenance scheme & estimates after reporting observations with suggestion for remedial action to Chief Maintenance Manager

v. Reporting any damage due to heavy rain, accident, earthquake, cyclone or any type of emergency.

vi. Supervision of the execution of the maintenance works

vii. Monitoring & approval of all safety aspects

viii. Necessary approval of inspection & maintenance works

ix. Coordination with various agencies like traffic police, local administration specialised execution agencies etc., in case of emergent repairs interruption to traffic by road blockage, etc.

x. Finalizing the execution report.

xi. Preparation of the monitoring report

Duties of Maintenance Engineer (one each for bridge, road, electrical components, ROB)

i. Conducting detailed inspection, surveys and site investigations as per approved format

ii. Prepare inspection report

iii. Analyse the cause of defects, structural inadequacy

iv. Prepare maintenance scheme & estimates after Reporting observations with suggestion for remedial action to Executive Maintenance Manager

v. *Reporting any damage due to heavy rain, accident, earthquake, cyclone or any type of emergency.*

vi. *Supervision of the execution of the maintenance works*

vii. *Preparation of the execution report.*

viii. *Arranging men, materials and machinery in advance as per requirement*

Duties of Inspector

i. *Arranging men, materials and machinery in advance as per requirement*

ii. *Conducting detailed inspection, surveys and site investigations as per approved format*

iii. *Reporting any damage due to heavy rain, accident, earthquake, cyclone or any type of emergency to the Maintenance Manager.*

iv. *Assist the execution of maintenance works*

Duties of Supervisor

i. *Arranging men, materials and machinery in advance as per requirement*

ii. *Supervise the execution of maintenance works*

iii. *Assist detailed inspection, surveys and site investigations as per approved format*

iv. *Reporting to the Maintenance Manager.*

Duties of System In-charge

i. *Routine inspection of the whole system*

ii. *Quick action to be taken in case of any type of fault*

iii. *Study the condition of whole system*

iv. *Supervise the execution of maintenance works*

v. *Reporting to the Maintenance Manager*

Duties of Electrician

i. *Daily inspection of each electrical unit*

ii. *Quick action to be taken in case of any type of fault*

iii. *Supervise the execution of maintenance works*

iv. *Reporting to the Maintenance Manager & System in-charge*

A 7.2 Training

Training of all the personnel involved in maintenance is an integral part of the maintenance function. Such training, as may be required by the operating and maintenance personnel to achieve better performance can be classified as below:

i. *Training* of *Managers*, Inspector and Supervisor.

ii. *Training* of Assistant, Electrician and gang men/mates.

(a) *Training of Managers, Inspector and Supervisor*:**-** These officers are either diploma holders or fresh degree holders. Either way they are expected to perform all the necessary management tasks sometime without any formal management orientation or training. The typical section officer learns about maintenance management while he is on the job. Frequently the new officer handles the job in the same manner as the person before him. Techniques which may or may not have worked twenty years ago are tried again today. Manager and other officers must be provided with the opportunity to attend maintenance management and operations training. The areas to be covered during the training should be

i. *Identification* of defects

ii. Planning *maintenance* operations effectively

iii. Crew *scheduling* and control to achieve higher productivity at low cost consistent with quality

iv. Problems of crew supervision—communicating and coordinating

v. Preparing *road* inventory

vi. *Management* by objectives

vii. Repair *methods*

(b) *Training of Assistant, Electrician and gang men/mates*: The training of maintenance Assistant, Electrician *and* gang men/mates is very essential in two areas like (i) Safety aspects and (ii) Repair methods.

The training in safety aspects will include:

i. Training of flag-men

ii. Use and lay out of temporary traffic signs for repair

iii. Safety oriented handling and parking of machinery and equipment.

The training in repair methods will include:

i. Proper patching methods

ii. Proper grading of shoulders

iii. Maintenance of drainage structure

iv. crack sealing

v. Painting and maintenance of the traffic signs and warnings

vi. Equipment maintenance.

Annexure -A.8

Checklist for Inspection of Extradosed Cable System

A. **Details about the Bridge and Extradosed Cable System**

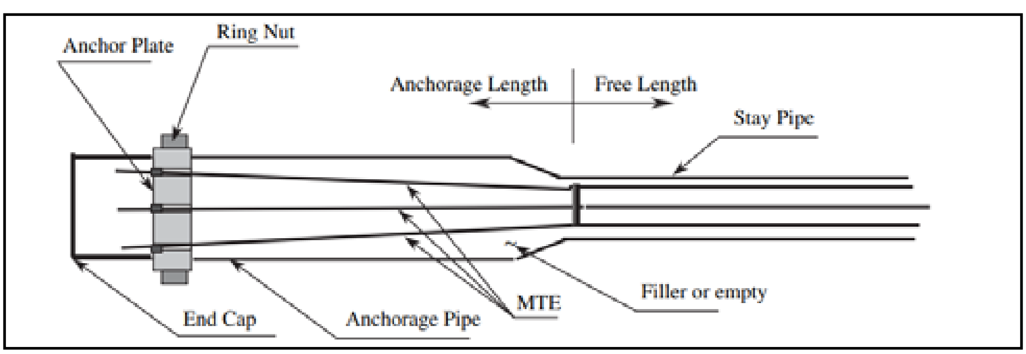
1. Bridge name and location
2. Year built
3. Main span length (m)
4. Side span length (m)
5. Type of extradosed cable system used?
6. Type of anchorage used?
7. Do cables go over "saddles" on the pylons?
8. Type of cable sheathing used
9. Do the cables have viscous dampers installed at deck levels?
10. Whether Coating/treatment has been provided on Cable within free length of cable? No/ Yes
11. If Yes, Type of coating/treatment-
12. Are the coatings/treatments on cables discontinued or removed within the anchorage zone? yes, no, not known, not applicable)
13. Whether any of grout has been used? Yes / No
14. If grout has been used, type of grout –
15. Whether any filler material has been used in the anchorage zone- Yes / No
16. If yes, type of filler material used-
17. Have the cables on this bridge (or any of their components) been repaired? If Yes Explain
18. Whether the strands or cables be replaced if needed?

B. Record of distresses noticed during Inspection

19. Have any corrosion noted in Stay cable
20. Have any of the cables subjected to rain-wind-induced oscillation?
21. Has cracking of the cable sheathing or sheathing connections been noted?
    (a) Any longitudinal cracking of sheathing?
    (b) Any transverse cracking of sheathing?
    (c) Any Excessive bulging of sheathing?

(d) Any damage of sheathing at the connection of damper

22. Has cracking or misalignment of the guide pipes been noted?
23. If there is protective tape wrapped around the cable sheathing, whether there is any deterioration of the tape? (tear, crack or delamination)
24. Have any problems associated with neoprene boots been noted?
25. Any cable sag noticed?
26. Identification of damage to connections between anchorage pipes and cable sheathing.
27. Inspection for damage, loosening, lack of water tightness, and deterioration of neoprene boots and band clamps
28. Inspection for damage or dislocation of neoprene rings and keeper rings, if applicable.
29. Identification of gaps between the neoprene rings and the sheathing.
30. Examination of sheathing surface inside the guide pipe through a boroscope or other means, looking for damage or deformation to the sheathing near the anchorage.
31. Review of cracking or damage to guide pipes or evidence of the impact of cable components on guide pipes.
32. Examination of surface conditions on the visible anchorage components including ring nuts, end caps, and bearing plates.
33. Examination of visible parts of saddles for damage, corrosion, and cracking, if applicable.
34. Review of evidence of moisture or fillers (such as grease) exiting the anchorage components. If there is an access port at the end cap (ideally at the lowest point), it can be opened and examined for moisture or moisture-contaminated grease
35. After the removal of the end caps on the sockets, visually inspect the anchorage plate and anchorage devices and to see if there is moisture or corrosion inside.
36. Examination of dampers, if any, as per recommendations of manufacturer.



Annexure-A.9

Design Aspects of Structural Health Monitoring System and Safety Evaluation

A.9.1 Designing a SHM

The following important aspects shall be considered while designing a SHM

(i)When designing a structural heath monitoring system, it is fundamentally important to design as an integrated system, which has all data flowing to a single database and presented through a single user interface.

(ii)The integration between the different sensing technologies can be simultaneously installed on the structure, for example, electrical strain gauges, fiber optic sensors, vibrating wire sensors, weather stations and corrosion sensors, can be achieved at several levels.

(iii)Different sensors can be connected to the same data acquisition system or otherwise, several data-loggers can report to a single data management system, which is generally a personal computer installed either on site or at a remote location.

(iv)Even though many of the sensors and systems provide their own software for data management and presentation, one of the goals of the SHM design shall provide a single integrated interface that does not require the end user to learn and interact with different user interfaces.

A.9.2 Functional requirements

There are several functional requirements that must be considered when designing SHM system:

(I) The system must implement advanced techniques, good performance, long term stability, and economic value rationality.

(ii) The system must have the capability to transmit data, process view, archive document and share long distance information.

(iii) The system must be able to collect data synchronously, real-time, long term, and hierarchically.

(iv). The system must have capability to assess, control, and calibrates itself.

(v) The system must be able to identify damages and evaluate structural health.

(vi) The system must be reusable and upgradable

A 9.3 SHM Framework

A Structural Health Monitoring (SHM) System is composed of the following modules:

1. Sensory System
2. Data acquisition and transmission system
3. Data processing and control system

4. Structural health evaluation system
5. Structural health data management system, and
6. Inspection and maintenance system (only if SHM is integrated with inspection and inspection)

Modules 1 and 2 are sensors, data loggers and cabling networks for signal collection, processing and transmission; Modules 3, 4 and 5 are computer systems assigned with different functions of system control and system operation. Module 6 has the purpose of performing the inspection and minor maintenance works for modules 1 and 2.

Sensors provide the basic data for a structural health detection and monitoring system. Their performance directly determines ultimate success of the detection and monitoring methods. Sensors need to record the structural mechanical conditions, and directly transform the measured parameters like strain, displacement, acceleration into signals for output. They must sense the variation of external environmental conditions, which needs sufficient reliability, sensibility and high reaction velocity to reflect the external information promptly and accurately.

Usually, the selection of sensors takes the following factors into consideration: the type, precision, resolution, frequency response and dynamic range; the distribution positions, the influence degree of the surrounding dynamic environment and measurement noise.

For the design of sensor modules and sensing technologies for extradosed bridges, three aspects should be considered: the parameter to be monitored type, the sensor type, and the positioning of the installed sensors.

For the selection of sensor systems, the following aspects should be taken into account:

(i)operational bandwidth; (ii)magnitude and frequency response over that bandwidth; (iii)sensitivity and accuracy; (iv)power supply requirements; (v)physical characteristics (dimensions, weight and material) (vi) environmental operating conditions such as temperature ranges; type of output signal; any signal conditioning requirements; and costs.

The commonly used sensor types and requirements are listed in **Table A9.1**.

Table A.9.1 Typical Sensor Types and Requirements

Monitoring Category	Specific Parameter/ sensor	Sensor requirement Accuracy, range, resolution, sensitivity
Structural global response monitoring	Acceleration/ accelerometer	Large span bridge with low-natural frequency: Frequency response range: 0 ~ 100 Hz Range ≥ ±2 g; transverse sensitivity < 5%

Structural local response monitoring	Strain measurement/ Strain guages	30% full range < static strain measurement < 80% full range Range of dynamic strain gage ≥ 2 × maximum predicted value
	displacement Measurement using displacement sensors	LVDT with measuring ranges between ±1 mm and ±50 mm and a quasi infinite resolution within a temperature range between –20°C and +120°C
	Temperature measurement/ Surface mounted vibrating wire guage	Structural surface temperature: Year extreme minimum temperature −20°C ≤ range ≤ year maximum temperature + 20°C ; Accuracy ≥ ±0.2°C ; resolution ≥ ±0.1°C
Environment monitoring	Wind speed and wind direction monitoring (3 cup Anemometer)	Range ≥ design wind speed of its installation height
	Wind pressure measurement	−1,000 Pa < range < 1,000 Pa Measurement error ± 2Pa
	Atmospheric Temperature monitoring	Atmospheric temperature: Year extreme minimum temperature −20°C ≤ range ≤ year maximum temperature + 20°C; Accuracy ≥ ±0.5°C ; resolution ≥ ±0.1°C
	Humidity measurement	Range: 0–100% RH; accuracy ≥ 3% RH

Data Section

The data section is divided into four parts: collection, transmission, processing, and management. **Table A.9.2** deals with the importance aspects to be considered with respect to data related processes.

Table A.9.2 Data Requirements

Data	Aspects/parameters	Requirements
Data collection	Selection of equipment	Charge signal: charge amplifier Digital signal: RS485 or compatible with sensor Data collection Analog signal: 4–20 mA and +5 V
	Sampling frequency	Load and environment monitoring: minimum value - Pier acceleration: 50 Hz; wind pressure: 20 Hz; earthquake: 50 Hz; temperature: 1/600 Hz; humidity: 1/600 Hz; rainfall: 1/60 Hz
	Time synchronization Accuracy	Same type of monitoring variables: <0.1 ms Different same type of monitoring variables: <1 ms
Data transmission	Transmission distance	Relatively short and without strong electromagnetic interference: analog signal Far or strong electromagnetic interference: digital signal Up to a few kilometers or even tens of kilometers: optical fiber transmission
Data processing	Pre-processing	Digital filtering, denoising, intercepting
	Post-processing	Specialized design
Data management	Functional requirement	quick display, efficient storage, report generation, and data archiving

A 9.3 Structural Safety Evaluation Requirements

The monitored parameters (load, environment, structural global parameters) can be used for (i) Early Warning and(ii) Safety evaluation of bridge which are discussed below.

Early Warnings

When the structural health monitoring system reports an abnormal situation (as discussed in Table A.9.3) to the concerned department, when the global monitoring data exceeds the threshold or limiting value. The threshold value is set according to different levels of danger to the bridge operating environment and structural components

Table A.9.3 SHM data for Early Warning

Safety Warning	Yellow and red warning	(i)Total weight of the vehicle or axle weight > design vehicle load × 1.5 → yellow (>design vehicle load x 2.0 → red) (ii)Maximum average wind speed > design wind speed x 0.8 → yellow (>design wind speed → red) (iii)Maximum/minimum temperature, maximum temperature difference, and maximum temperature gradient > design value → yellow (iv)Displacement > design value x 0.8 → yellow,(>design value or more than 10 yellow warnings appear within a month → red

Safety Evaluations

There are two levels of structural safety evaluation- First level and Second level. Based on the analysis of structural health data collected (Table 11.6) first-level safety evaluation shall be carried out.

In the first level safety evaluation, the eigenvalues of the structure are calculated using the monitoring data, and compared with the predetermined values. If the measured and computed vibration parameters are close, then the bridge is structurally safe. If the measured frequency values are lower than the computed values, then it may indicate a stiffness degradation and thereby indicating the need to perform second-level safety evaluation. Also, if the monitored global response parameters as indicated in Table 11.7 are abnormal, second-level safety evaluation should be conducted.

Second-Level safety evaluation is a structural safety assessment method in which the corrected structural finite element model, monitored load, and design load are used for structural re-analysis, ultimate load carrying/bearing capacity analysis, and the evaluation of both the bridge structural safety state and the estimated safety reserve.

If the structural local response is found abnormal, special investigations /assessment should be performed. Some of the situations under which special investigations are required are as follows- (i) Measured maximum average wind speed is greater than the design wind speed (ii) barge/ship impact of the bridge (iii) bridge is flooded.

If bridge damage is found during the special assessment, the second-level safety evaluation should be carried out based on the monitored data and special investigations results.

Table A.9.4 Global Response Parameters

Structural Health Assessment	**Type of Monitoring**	**Structural performance/ load & environmental Parameters**
Data Analysis	Structural global response monitoring	(a) Acceleration: absolute maximum, maximum root mean square value (b) Deformation: mean, absolute maximum (c) Displacement: maximum, cumulative displacement (d) Modal parameters: frequency, mode, and damping ratio
	Structural Local Response Monitoring	Strain: mean, maximum, minimum, stress amplitude maximum; support reaction time course: mean, maximum/minimum, and maximum change
	Load and environment	(a) Vehicle load parameters: bridge traffic, vehicle type, axle weight, gross weight, speed, and overload ratio (b) Wind parameters: speed, direction (c) Earthquake data: acceleration peak, velocity peak, duration, spectrum, and response spectrum (d) Temperature data: maximum/minimum temperature, the maximum temperature gradient of the section (e) Humidity data: maximum internal/ external humidity of the component, mean value, and time exceeding the limit (f) Rainfall data: maximum rainfall per hour, cumulative rainfall