5.6 Quality Control during Construction

5.6.1 *Accuracy Management of Entire Bridge*

In the cantilever construction of cable-stayed bridges, the management is necessary not only to ensure the determined configuration of bridge such as vertical alignment in completion stage with a specific accuracy range, but also to control the deflection of girder, prestressing force of stay cable and stress of member in each construction stage. In addition, special attention is important for accuracy management of inclination of pylon. Since deflection of girder, inclination of pylon, prestressing force of stay cable stress of member all have a close and interdependent relationship mutually, a comprehensive management method is essential. In contrast to other girder bridges, cable-stayed bridges have less stiffness, and the prestressing force of stay cable and deflection of girder are more influenced by change of temperature. Therefore, it is important to carry out the accuracy management with clearly grasping the margin for the prescribed limit values in each stage of construction.

In addition, as concerns the cable-stayed bridges, the expansion of girder is also an important issue in accuracy management. The amount of expansion includes the elastic contraction due to prestressing force of stay cable, the contraction caused by creep and shrinkage, the expansion and contraction by change of temperature. In particular, it is necessary to consider in advance the manufactured length of cables. Furthermore, if the width of girder is wider, deflection control in the transverse direction may be required. Also, in case of skew bridge and curved bridge, torsion control may necessary. Thus, these cases also need consideration.

The construction management of cable-stayed bridges may need tension adjustments after the closure of center of span. Although there is often a difference between the calculated deflection and the actual deflection, as concerns the cable-stayed bridges, it is adjustable by adjustment of cable force not only during cantilever erection but also after closure.

The following items are usually included in the construction management of cable-stayed bridges.

- ✓ Preparation of a detailed data base; detailed execution plan (deflection, expansion and contraction) on each construction stage and calculated value influenced by various factors.
- ✓ Measurements at the construction site ; clearly grasping the deflection of girder, inclination of pylon, prestressing force of stay cable and stress of member at every stage of construction.
- ✓ Data processing and analysis; analyze the cause of difference comparing measurement values collected at the construction site with design planned values. The influence values are reviewed and any relevant adjustments of prestressing force might be made.

As the above items are done at the construction site, and the amounts of data to be dealt with are large and complex, it is standard to establish a management system where computations on the basis of data is made available to site for further action at the earliest.

The basic items measured at the construction site are shown in the **Table 5.2**.

In the actual construction of bridge, the followings are thought as factors which cause the difference comparing with design planned values: variation of material characteristic, difference in estimation of shrinkage and creep, difference in loading of construction equipment, difference in deflection and elongation by temperature, and other factors in construction. In order to minimize the difference possibly caused by these factors and to ensure the configuration and stress condition as designed

in completion stage, it is important to use the construction management system and enhance the management quality for each construction step.

Table 5.2 Items to be measured in Construction Management System [2]

Item	Place	Measurement Objectives
Deflection	*Girder*	Obtain data on deflection of the girder during construction to be used for deflection management.
Expansion	*Girder*	Regards the expansion amount of girder to use in length control.
Tension	*Girder*	Used for tension management during cable tensioning. Get data on tension changes during construction to be used for tension management.
Temperature	*Girder* *Pylon* *Cables Air Temperature*	Grasp member temperature changes caused by sunshine and investigate the effect of temperature on girder deflection and cable tension, gather data for correction.
Stress	*Girder* *Pylon*	Obtain data on stress changes on the main girder and tower during construction to be used for management.
Inclination	*Foundation* *Pier* *Pylon*	Obtain data on foundation, bridge pier and pylon inclination caused by unbalanced moment, investigate their effects on girder deflection.
Settlement	*Foundation*	Obtain data on foundation, caused by incremental loading, unbalanced moment, investigate their effects on girder deflection.

It is necessary to determine the range of management values for construction management in terms of not only structure safety in construction or completion stages, but also serviceability during in service stage and stress condition of members. **Table 5.3** shows examples of general management indicators for the main girder, pylon and stay cables. In addition to checking the indicators and confirming them with the design conditions and construction standards, it is necessary to consult with the owner of the bridge to confirm the process.

Table 5.3 Construction Management Indicators [2]

Deflection of Girder	The level which does not produce any problem for vehicle running during the service time.
Inclination of Pylon	The level which does not produce any trouble due to additional moment.
Introduced Cable Tensions	The level in which member stress is within the limit values

5.6.2 *Quality Control of Concrete*

Generally, very high-strength concrete is used for cable-stayed bridges and extradosed bridges. Therefore, thermal cracking should be investigated in relatively massive parts such as anchorage of stay cable and base of tower.

Moreover, it can be considered that crack occurs in girder during cantilever construction because the existing slab deck restrains the new slab deck's thermal and shrinkage deformations. Since the

girder of cable-stayed bridges and extradosed bridges usually has wide cross-section, this phenomenon becomes more pronounced. Therefore, reinforcement should be arranged based on thermal stress analysis carried out beforehand.

In the girder, the HTSs and reinforcing bars at the anchorage of stay cable and top of column are placed in highly condensed arrangement, and the concrete is usually placed by using concrete pumps. Thus, adequate slump should be determined by considering the filling and workability of concrete.

Since the reinforcement of the pylon, where the stay pipe for stay cable anchorage or saddle, reinforcing steels, steel frames for assembly are arranged, are highly condensed and concrete generally need to be pumped to high altitude, the slump that is necessary for pumping performance should be determined. Also, concreting method shall be examined by considering sufficiently the capability of carrying facility based on the relationship of factors such as characteristic of concrete, height of pylon, quantity of one-time placed concrete. If necessary it is advisable to do initial mockups to study the construability of the elements.

5.6.3 *Quality Control of Steel Members*

With regards to quality control of steel member, for example, in case of composite bridge with steel girder beams with concrete deck, or mixed girder where steel girder is used in the center span, or pylon's anchorage structure made of steel shell, or a steel deck or steel pylon itself, reference should be made to IRC:SP:104. "Guidelines for Fabrication and Erection of Steel Bridges" Moreover, in usage of steel members, consideration is necessary to ensure accuracy required by trial-assembly works in the factory and pre-assembly at construction site prior to erection.

5.6.4 *Prestressing Management of Stay Cable*

Stay cable alignment has very small change of angular which is similar to external cables, and loss of prestressing force by friction is also small. Therefore, as a general rule, the method to control relation between cable force and elongation should be applied for the prestressing management of stay cable. In addition, prestressing management requires the sufficient understanding of the characteristics of anchoring system of stay cable (anchorage type and saddle bent type) and method of fabrication (site-fabricated and pre-fabricated).

5.6.4.1 Prestressing Management for Separate Fixing type.

In the prestressing management for separate fixing type, prestressing management should be done dealing with tension force primarily while elongation secondary. There are two methods for initial tensioning. In one method, all PC strands are inserted at once and tensioned all together using center hole prestressing jack. In other method, PC strands are inserted and tensioned one by one using hydraulic single strand jack. After the initial prestressing during construction, it is necessary to adjust the tension appropriately in order to improve stress conditions of girder, pylon and pier, and to correct the final configuration of the structure. Generally, hydraulic jacks are placed in symmetrical position in the pylon or girder. Moreover, cable tension can be measured by using load cells that are attached in each cable. And also, it is found indirectly from the vibration frequency of cable obtained by using accelerometer to vibrate cable intentionally.

5.6.4.2 Prestressing Management for Through Fixing Type.

For prestressing management for through fixing type, since the tensioning is conducted from both ends of the girder and loss of prestressing force by friction (μ is small, prestressing management should be primarily done with tension force while elongation is secondary. However, the form and

friction coefficient of the saddle shall be considered in calculation of elongation. The Influence of this Friction Coefficient can follow *"Guidelines for the use of External and Unbonded Prestressing Tendons in Bridge Structures (IRC:SP:67)".* Generally, prestressing should be performed collectively so that elongations at both ends are equal in order to apply the required cable force to all cables that are used in the same stage of installation. In this type of cable, if designed tensions at both sides of saddle are not same, the slipping of cable at the saddle should be checked and the required tension should be secured.

5.6.5 *Acceptance Inspection and Storage Management Plan of Materials*

Materials that are used for stay cables shall be checked whether it satisfy the required standards and specifications on acceptance. Also, scratches, twists, rusts shall also be checked by visual inspection. Therefore, it is necessary to establish management plan regarding acceptance inspection of material and storage procedure prior to the commencement of construction.

Since the saddle and steel anchorage structure require high accuracy in fabrication, quality control plan shall be prepared prior to fabrication, and acceptance inspection of material including factory inspection shall be performed.

In addition, the "Recommendations for Stay Cable Design, Testing and of U.S. Post-Tensioning Institute (PTI) can be referred. This document presents considerations regarding on-site acceptance inspection and quality control of components of stay cable.

- ✓ The components of site-fabricated cable; diagonal cable (HTS), anchorage, stay pipe and filling materials, are transported separately to the site in accordance with the progress of installation. Therefore, acceptance inspection for each lot is executed immediately after arrival of materials. The transported materials are inspected by collating the mill certification report, delivery statement and the detailed order sheet. Also, acceptance checklist is used to confirm that scratches, twists, rust are not found. The results are saved after recording it in a check sheet.
- ✓ Adequate storage and curing should be done in order not to impair the quality of materials during storage. Since these materials are generally heavy and stay cables are packed in drum form, it is necessary to maintain safe storage conditions by considering over-turning and rolling during temporary setting.

CHAPTER - 6
LOADS AND LOAD COMBINATIONS

Section I: Loads

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

6.1 Dead Load

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

Stay Cable Weight

For stay, the weight of the main tension element, and any additions to the stay, such as sheaths, grout, etc., shall be included as dead load in the stay. The Self weight of the stay cable and its connection including sheathing shall be as per the recommendations of the manufacturer.

In the absence of the any data following can be used:

Weight of parallel wire / parallel strand = A_s . n . ρ_s

where

A_s - Nominal area of wire / strand,

ρ_s - Density of the steel 7850 kg/cum

n – number of wires / Strands

The Weight of the protection material viz. Heavy galvanization, Sheath, grout can affect weight of the cable and increase its weight by 10% to 25%. **Fig. 6.1** shows example of a 7-strand prefabricated cable (GJ15-7), having weight 9.79 kg/m (i.e. about 25% additional weight of protection)

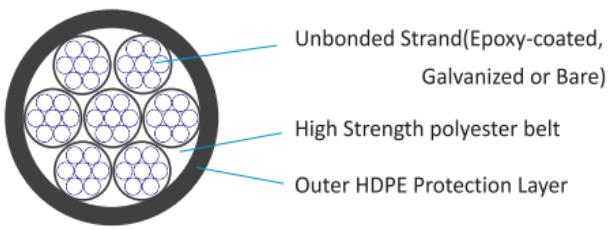


Fig.6.1- Strand Prefabricated Cable (GJ15 - 7),

6.2 Prestressing Force in Concrete

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

6.3 Secondary Effects

In the design of cable-stayed bridges the effects of shrinkage and creep of concrete shall be considered as per the IRC:112 and load factors and combinations as per IRC:6.

6.4 Superimposed Dead Load

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

6.5 Surfacing 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 existing coat, and possibility of change in type of surfacing. Partial safety factors and load combinations shall conform to provisions of IRC:6.

6.6 Snow Load

The effects of Snow Load, wherever applicable, shall be considered as per provisions of IRC:6.

6.7 Vehicular live load (Q)

6.7.1 *Vertical Loads Effects*

Following shall be considered for Vehicular Live Load:

i) Vehicular Live Load in accordance with **IRC:6 Clause No. 204.1**, including combinations of Live load according to Clause No. 204.3 shall be considered for the design.

ii) Vehicular Live Load with Congestion factor will be considered according to **Clause No. 204.4** of IRC:6. For congestion factor loading to be applicable, all the spans shall be loaded.

iii) Special Vehicle Loading shall be considered in accordance with **Clause No. 204.5** of IRC:6.

iv) Reduction factor for the Vertical effects of the live load shall be considered as per **Clause No. 205** of IRC:6.

6.7.2 *Impact factor due to Vehicular Live Loads*

Vehicular Live loads are multiplied with an Impact Factor to accommodate this dynamic effect in the design of bridges. The magnitude of the impact factor depends on the bridge span, stiffness and surface roughness, and vehicle dynamic characteristics such as moving speed and isolation system.

Impact factor shall be Considered According to **Clause No. 208** of **IRC:6**.

6.7.3 *Longitudinal Forces*

Longitudinal force caused by tractive effort of vehicles or by braking, and / or Longitudinal force caused due to frictional resistance offered by free bearings due to change in temperature, or due to creep or shrinkage, shall be considered in accordance with **Clause No. 211** of IRC:6.

6.7.4 *Centrifugal force of Vehicles Travelling at High Speed along a Curved Path*

Shall be considered in accordance with **Clause No. 212** of **IRC:6**.

6.8 Pedestrian Load/Foot Path Load

Shall be considered in accordance with **Clause No. 206** of **IRC:6**.

6.9 Fatigue Load

The performance of Stay cable is dependent on its fatigue performance under repeated live load cycles. However, this will depend on the magnitude of the live load as a proportion of the dead load.

For all concrete / steel components of the cable stayed bridge, excluding the stay cables and its connections, provision of IRC:6 for fatigue load and relevant annexure of IRC:112, provision of IRC:22 and IRC:24 shall be applicable. For stay cable design, provisions given herein shall be followed.

All cable stayed bridges need to be designed for fatigue limit state as explained in **Chapter-7** for design.

Fatigue Truck to be considered shall 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 shall be used for the fatigue assessment with the fatigue load so positioned as to have worst effect on the detail or element of the bridge under consideration. In case of divided carriageway, single passage of fatigue load shall be considered for each carriageway. The Fatigue truck shall be placed on the carriageway with respect to the kerb accordance with IRC:6. In case of divided carriageway, single passage of fatigue load shall be considered for each carriageway.

6.10 Thermal Loads

All steel and concrete bridges shall be designed for IRC:6 requirements for uniform temperature ranges and thermal effects. In addition, non-uniform temperature variations shall be considered. Preference shall be given to using a light colored 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 shall 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 that 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 shall be used for brightly coloured HDPE pipe, and black HDPE pipe, respectively. Similarly, 5°C linear temperature difference between the opposite faces of the pylon shall 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 shall be neglected

6.11 Cable replacement

The design of cable-stayed structures shall provide for the replacement of any individual 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 shall be considered for the design. Some cable systems offer possibility of replacing cables strand by strand. In such situations, replacement method shall be clearly indicated in the Maintenance Manual.

6.12 Cable Breakage

All cable-stayed bridges shall 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 shall be 1.5 (dynamic amplification factor) times the static force in the cable. DAF of 1.5 is recommended for parallel strand cable and parallel wire cables. This force shall be applied at both the top and bottom anchorage locations in direction opposite to the cable tension.

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

6.13 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 α_{SLS}.

The transverse support provided by the guiding device to the cable causes a kink in the cable geometry, **Fig. 6.2**, and a corresponding transverse load is applied by the cable on 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 by 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 shall 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 preliminary design of the structure supporting the guiding device installed in a cable, an angular kink of α = ± 1.4 degrees (± 25 mrad) is suggested as a reasonable assumption for SLS. Careful detailing for 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 shall 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:

6.13.1 *Ultimate Limit State*

$F_{ULS} = \gamma \alpha_{SLS} F_{SLS}$

where,

$\Delta\alpha_{SLS}$ corresponds to ± 25 mrad

γ - Load Factor 1.5

F_{SLS} – Max Tension in cable under Rare Combination

6.13.2 *Serviceability Limit State*

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

6.13.3 *Fatigue Limit State*

The following can be considered:

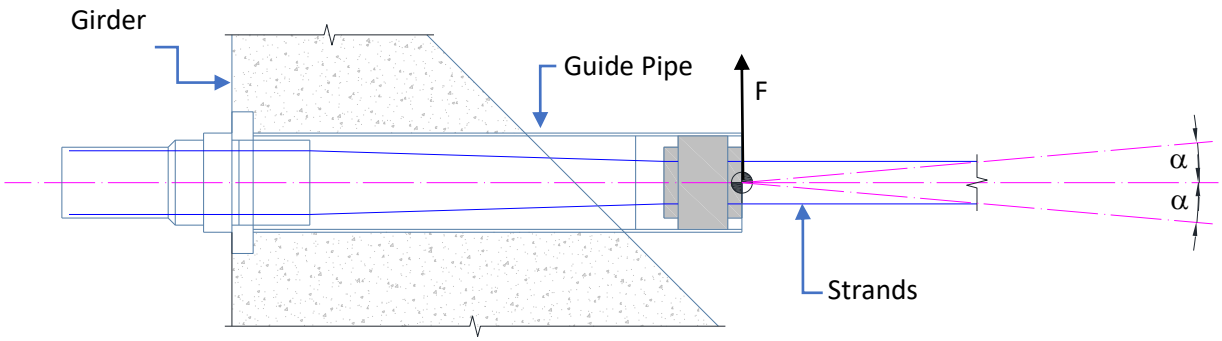
$F_{FLS} = \Delta\alpha_{FLS} F_{perm}$

where,

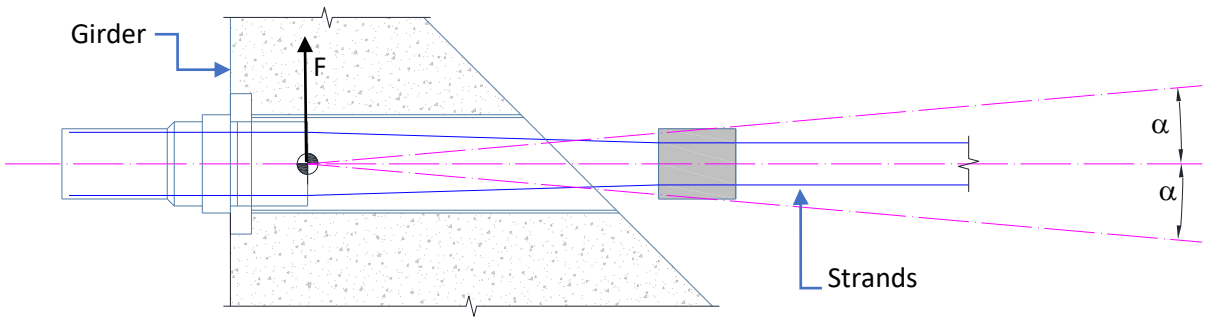
F_{perm} Tension in cable corresponding to the service loads under frequent load combination

$\Delta\alpha_{FLS}$ 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. 6.2 Transverse Force by Cable Angular Deviation

6.14 Bending Stresses in Cables

Fib 89 **[1]** provides guideline 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 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 shall be calculated by the designer. When using cable systems designed and tested in accordance with Fib 89 **[1]** and this guidelines, fatigue angular deviations applied at the anchorage or saddle up to ± 0.6° (± 10 mrad) with 2 million cycles and service angular deviations up to ± 1.4° (± 25 mrad) with 100'000 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.

6.15 Fire, impact, vandalism

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

6.16 Wind Loads

For wind load and design provisions refer **Chapter 9** and **Chapter 2** of this guideline.

6.17 Seismic Loads

For seismic design provisions refer **Chapter 8** and **Chapter 2** of this guideline.

6.18 Cable forces

The cable forces are of two types i.e,

1) cable force at the end of the staged construction which include dead load as well as the 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 shall be arrived directly from the analysis of each action.

6.19 Loads during Construction.

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

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

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

Section II: Load Combinations and Load Factors

For the design of the elements of the cable stayed bridges (viz, Deck, Pylon, Pier Foundations, Stays) load combinations as per IRC:6 for Stability, Serviceability and Strength shall be followed. Additional combinations in respect of cable stayed bridges as given below shall be considered.

6.20 Combinations for SLS Check

Table: 6.1 Partial Safety factor for Verification of Serviceability Limit State for Stay 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:				
Adding to effect of variable loads	1.2	1.2	1.2	1.2
d) Relieving the effect of variable loads	1.0	1.0	1.0	1.0
Settlement Effects:				
Adding to Permanent loads	1.0	1.0	1.0	1.0
Opposing to Permanent loads	0	0	0	0
Variable Loads:				
Live Load		*		**
Leading Load	1.0	1.0	-	1.0
Accompanying Load	0.75	0.75	-	0.75
with Congestion factor	No	Yes	No	No
Special Vehicle Loading	-	-	1.0	-
Braking	1.0	-	-	1.0
Centrifugal	1.0	-	-	1.0
Wind Loads				
Leading Load	1.0	1.0	-	-
Accompanying Load	0.6	0.6	-	-
Live Load surcharge	0.8	0.8	-	-
Thermal Loads				
Leading Load	1.0	-	-	1.0
Accompanying Load	0.6	-	-	0.6
Cable Load	1.0	1.0	1.0	1.0

* Live Load without transverse eccentricity

** Reduced Live Loads Lanes

6.21 Combinations for ULS check

Table: 6.2 Partial Safety Factor for Additional verification of Stay Cable Strength

Loads	Basic combination	Basic combination (Congestion factor)	Basic combination (SV Loading)	Cable Break (Accidental)
Permanent load: Dead Load, SIDL except surfacing				
Adding to effect of variable loads	1.35	1.35	1.35	1.0
Relieving the effect of variable loads	1.0	1.0	1.0	1.0
Surfacing:				
Adding to effect of variable loads	1.75	1.75	1.75	1.0
Relieving the effect of variable loads	1.0	1.0	1.0	1.0
Earth Pressure due to Back fill:				
Leading Load	1.50	1.50	1.50	1.0
Accompanying Load	1.0	1.0	1.0	1.0
Variable Loads:				
a) **Live Load**				
b) Leading Load	1.5	1.5	-	0.75
c) Accompanying Load	1.15	1.15	-	-
d) Construction live load	1.35	1.35	-	-
with Congestion factor	NO	YES	NO	NO
e) Special Vehicle Loading	-	-	1.15	-
f) Braking	1.5/1.15	-	-	0.75
g) Centrifugal	1.5/1.15	-	-	0.75
Wind Loads				
Leading Load	1.50	1.50	-	-
Accompanying Load	0.9	0.9	-	-
Thermal Loads				
Leading Load	1.5	-	-	-
Accompanying Load	0.9	-	-	-
Live Load surcharge	1.2	-	-	-
Erection load	1.35	-	-	-
Cable force (at the end of construction stage)	1.35	1.35	1.35	1.35
Accidental Effects:				
Cable break	-	-	-	1.1

6.22 ULS check for Stay cables:

Cable forces under ULS loads shall be calculated as explained below.

1) Cable forces at the end of the stage construction which includes dead load and cable shortening shall be considered with factor 1.35.

2) Cable force due to other loads (for e.g. 1.5 LL for basic ULS combination above) shall be arrived from factored load applied on the completed structure.

3) Both the cable forces (1) and (2) above shall be combined to arrive at force F_{ed} in cable under ULS combination

4) Force F_{ed} obtained above shall be verified for ULS as per **Cl. No 8.8.2** in **Chapter 8**

CHAPTER - 7

ANALYSIS AND DESIGN

Section I – Analysis

7.1 Introduction

The analysis of a cable stayed bridge is usually carried out using 3D frame method or finite element method by using various commercially available software or proprietary software developed by the design consulting firms. The capabilities of these software vary significantly and hence they are to be chosen judiciously depending on the intent of the analysis.

7.2 Structural Modelling

The models to be developed for the analysis also vary depending on the purpose of the analysis and stages of the design process, e.g. preliminary analysis for developing the concepts, feasibility, deciding the configuration & form of the bridge or detailed analysis for the final execution design.

Very often, different models need to be developed depending on type of loads e.g., seismic or wind loads. The model needs to be modified to carry out the construction stage analysis using the features available in the software. Sometimes, local analyses are required to be carried out in detail to investigate the local response of the structure, say at the anchorage or at other intricate locations.

Adoption of linear elastic static analysis or non-linear analysis (geometrical and/or material) shall be decided based on the size, configuration of the bridge and severity of the loads.

The choice of the type of elements to be used in the model, e.g., truss, beam, solid or shell etc., with their appropriate features available, is also an important decision which depends again on purpose of the analysis and the usage of the results of the analysis for the design.

Generally, simplified line element model of a bridge is adequate not only to understand the general behavior of the structure for the preliminary design, but also for seismic or aerodynamic analysis etc. A finite element model with solid and/or shell elements can be used for stress analysis for local parts of the structure, to have more insight. To trade off the computational efficiency and capability, one can establish a multi-element model to cater for the objectives as explained below.

7.2.1 *Line or frame element Model* ***(Fig. 7.1)***

In a line element model, components of a cable-supported bridge are modelled by line elements including beam elements, truss elements, and rigid links. Pylons and piers are usually modelled with beam elements based on their geometric properties. Cables are often modelled by either truss elements, or cable elements which take into consideration the geometric non-linearity due to cable sag; the static equilibrium profile of the bridge also

needs to be calculated iteratively, based on the tension forces in the stay cables and the weight of the bridge deck.

The bridge deck, however, is more challenging to model. Usually, there are two approaches that can be employed to simplify the complicated bridge deck into a series of beam elements.

In the first approach, the deck is modelled as a central beam (spine beam). The equivalent cross-sectional area of the deck is calculated by summing up all cross-sectional areas of the deck.

In the case of a composite section, the areas should be converted to that of one single material, according to the modular ratio. The neutral axes and moments of inertia about the vertical and transverse axes are also determined. In a similar way, the mass moment of inertia of the deck should include those of all members, according to their distances from the centroid of the section.

Constraints, usually spring elements, rigid links or direct coupling of nodal displacements, are necessary to connect different parts of the model together and to enforce certain types of rigid-body features. For example, the nodes of the deck, bearings and towers do not coincide with each other. Rigid links are usually used to restrain their motions in different directions, depending on the bearing types and boundary conditions. Rigid links are also used to connect the spine beam with cables in the spine beam model.

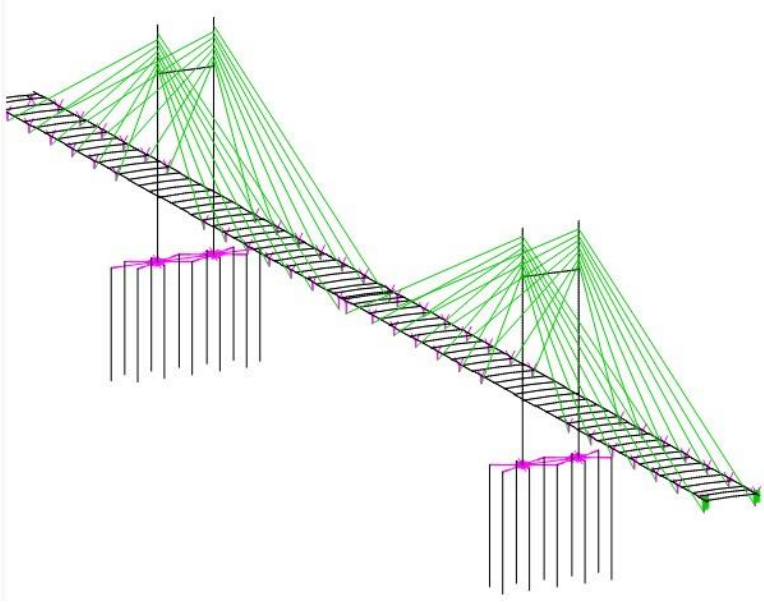
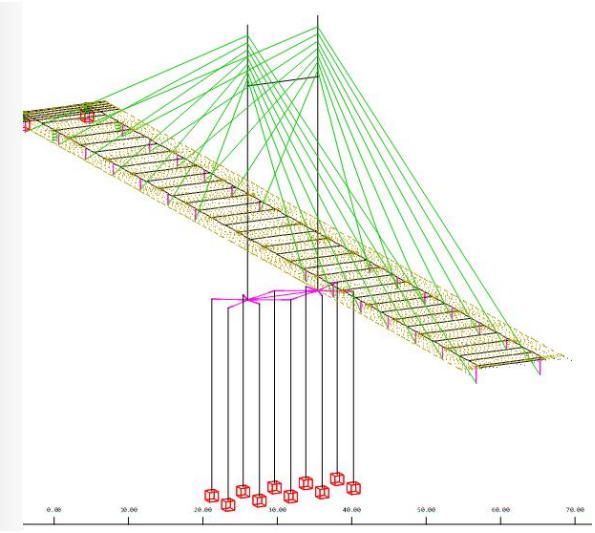


Fig. 7.1 3D Frame Model

7.2.2 *Multi-Element Model* ***(Fig. 7.2 & 7.3)***

To capture the local responses of some critical members and/or to obtain more accurate results, the entire bridge can be modelled with different types of elements. The ever-increasing capacity of computers makes this multi-element modelling more and more

popular. In such a multi-element model, the components of interest can be modelled with shell elements or solid elements and other components with line elements. Special care must be taken at the interface between a solid element, a shell element and a beam element, because they have different DOFs. Multi-point constraints (MPC) are needed to connect the different elements in order to avoid displacement incompatibility among the nodes. Various MPC equations for beam to shell, shell to solid and beam to solid connections have been developed in terms of total energy principle and implemented in the commercial software.



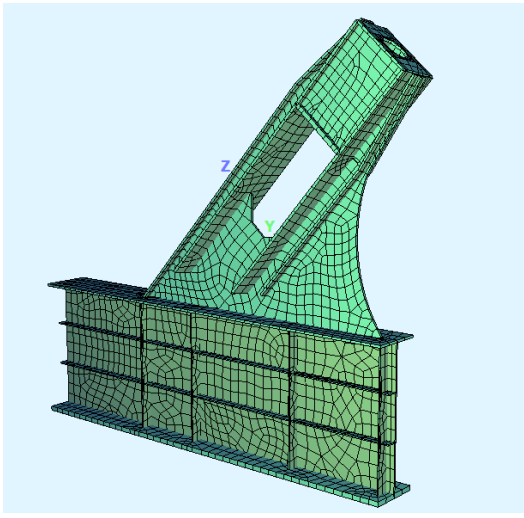


Fig. 7.2 Model with Line and Quad Element **Fig. 7.3 FE Model of Anchorage**

7.3 Geometric Nonlinearities

There are three main sources of geometrically nonlinear behaviour of cable-stayed bridges: the beam-column effect of the Deck as well as the Pylon, the large displacements (known as P-D) effect and the cable sag effect **[1]**. It is generally accepted that the latter is the most relevant of those and, therefore, its consideration is mandatory in cable-stayed bridge design, even for simplified models of such structures.

The Cable sag gives the most significant nonlinear effects and can be a decisive issue in the global behaviour of these structures. Cable modelling can be carried out using truss element (the modified elastic modulus element), cable element (the elastic catenary element) or multiple straight link discretisation. Element formulation shall include nonlinear solver to account for large displacement if the same is anticipated in case of longer cables.

Beam-column effect generally does not significantly contribute to the geometrically nonlinear behaviour of cable stayed bridges. Even though potentially important close to the ultimate load, neither moment-axial interaction nor bowing are important effects when compared with either cable sag or large displacement effect

For slender decks and pylon, Beam-column effect can be accounted either through the use of stability functions or by considering a narrow mesh refinement with a finite element solver accounting for large displacements.

7.4 Modelling Aspects

7.4.1 *Deck*

For the design of the structural elements in a cable stayed bridge it is sufficient in most cases to use a standard two- or three-dimensional line or frame analysis program.

For bridges with only one, central cable plane, a two-dimensional structural system is adequate to analyse the structure under vertical loading due to dead load and traffic load. This analysis gives the forces in the stay cables, and the axial forces, shear forces and bending moments in the girder and the pylons.

For the torsion induced in the girder under one-sided traffic load, the girder can be analyzed subsequently without taking the cable system and the pylon into account. The same type of analysis applies to the girder under lateral load, e.g. wind load.

In bridges with two cable planes the mathematical model has to be three-dimensional as depicted in **Fig. 7.4 (a)** i.e. with one beam element to model box girder and cable connected by rigid link element on both sides or **Fig. 7.4 (b)** i.e. with two longitudinal beam elements to model the main girders and a large number of transverse beam elements to model the transverse girders. The stay cables can in most cases be modelled as truss element carrying pure tension.

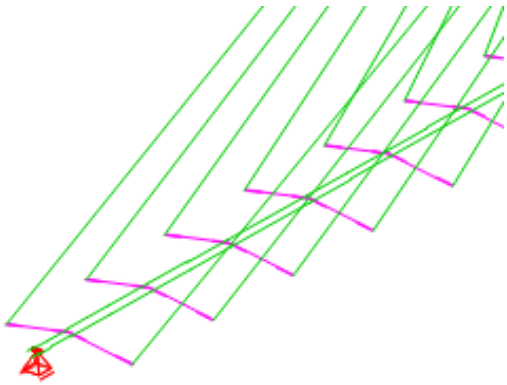


Fig. 7.4 (a) Analytical Model Twin Cable

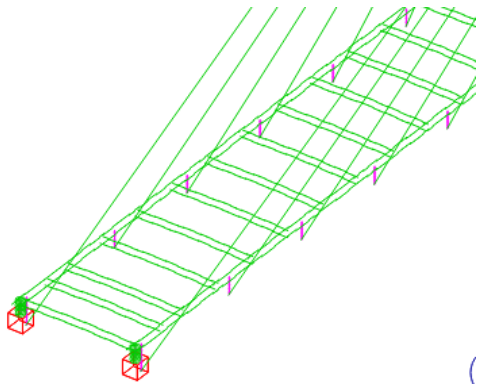


Fig. 7.4 (b) Box Girder Twin Cable with Plate Girder

For bridges with a single box shaped girder and two cable planes, as illustrated in **Fig. 7.4 (a)**, the mathematical model should comprise a single longitudinal girder with the flexural and torsional stiffness of the box shaped main girder. At the points of cable attachment, the central beam should be joined to rigid link elements, as illustrated in **Fig. 7.4 (a).**

7.4.2 *Cable and Stay Sags*

In the analysis, the stay cable is regarded as a straight member subjected to pure tension. In reality an inclined cable is always slightly curved due to the action of the own weight of the cable. For long stay cables, the sag tends to reduce the axial stiffness as the elongation is not only due to elastic strains in the cable wires but also due to reduction of the sag, as illustrated in **Fig. 7.5.**

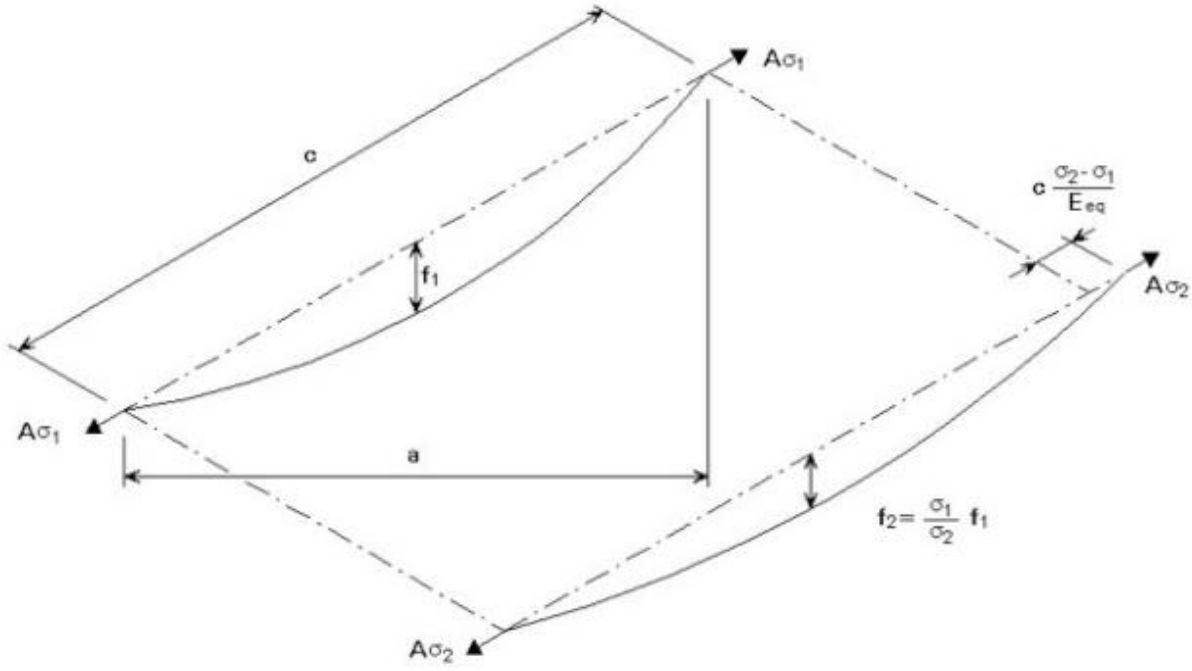


Fig. 7.5 Influence of Gravity on cable stiffness

For stay cables with horizontal projections up to 150 m and moderate stress variations, the sag effect can generally be disregarded, but for longer cables the stiffness of the cable support is overestimated if the axial strain is only considered.

The sag effect can be considered in the analysis by substituting the real modulus of elasticity E of the cable material by an equivalent modulus of elasticity E_{eq} determined by:

$$E_{eq} = \frac{E_{sc}}{1 + \frac{\gamma^2 a^2}{24}\left(\frac{\sigma_1 + \sigma_2}{\sigma_1^2 . \sigma_2^2}\right) E_{sc}} \qquad \text{(Eq.1)}$$

where,

γ - is the density of the cable material

a - the horizontal projection of the cable

σ_1 - is the initial dead load, stress

σ_2 - the final stress (from dead + live load)

E_{sc} - Modulus of Elasticity for stay cable material

Equation (1) which introduces a secant modulus, requires an iteration in the structural analysis, as the final stress σ_2 is unknown from the beginning.

For stay cables with moderate stress variations and horizontal projections up to 250 - 300 m, it is allowable to substitute the secant modulus of Equation (1) by a tangent modulus E_{tan} derived simply by inserting $\sigma_2 = \sigma_1$ in Equation (1):

$$E_{tan} = \frac{E_{sc}}{1 + \frac{\gamma^2 a^2 E_{sc}}{12\sigma_1^3}}$$

(Eq.2)

In the latter case an iteration can be avoided as the tangent modulus depends only on the initial stress σ_1.

When applying the more exact secant modulus approach, the first step in the iteration is often based on axial cable stiffnesses found from Equation (2), whereas the following steps are based on stiffnesses determined from Equation (1).

Presently, several commercial software are available which can calculate cable sag and effective stiffness of the stay cables during construction and service.

7.4.3 *Pylon*

The pylon is subjected primarily to compression from the vertical components of the cable forces, and can be modelled as line element. Second order effects in the pylon shall be considered in the design based on the **Section 8.5.3** of this Guideline.

7.5 Structural Analysis

7.5.1 *Linear and pseudo-linear analysis*

The linear analysis of the numerical model of cable-stayed bridge uses the standard beam and truss elements. Cable-stressing forces are applied as loads, by evaluating the shortening in length of each cable element.

Pseudo-linear analysis is a linear iterative method in which only the modified elastic modulus of the truss elements, used to model the stay-cables, is updated according to the current cable tension, to account for the cable sag effect. No geometry updating is made and so, as in linear analysis, out-of-balance forces are not evaluated. The relevant advantage of this approach is that it may be undertaken with common linear analysis software.

7.5.2 *Nonlinear Analysis*

Eurocode 1992-2 provided methodology for Non-linear analysis of concrete bridges, which accounts for the non-linear nature of the material properties. This includes the effects of cracking in concrete and non-linearity in the material stress–strain curves. The analysis then ensures both equilibrium and compatibility of deflections using these material properties. Such an analysis may be used at SLS and ULS. Non-linear analysis also models non-linearity in structural response due to the changing geometry caused by deflections. This is important in the design of elements where second-order effects are significant, as explained further in this guideline. It is for second-order calculation for slender members, such as bridge piers and pylon, that non-linear analysis is often particularly beneficial, as the simplified alternatives are usually quite conservative.

7.5.3 *Second Order Effects*

Second-order effects are additional action effects caused by the interaction of axial forces and deflections under load. First-order deflections lead to additional moments caused by the axial forces and these in turn lead to further increases in deflection. Such effects are also sometimes called P - Δ effects because additional moments are generated by the product of the axial force in the element and system deflections. IRC:112 provides simplified method and general method for obtaining second order effects.

The nominal curvature method can be used to obtain second order effects in pylon when the pylons have simple forms as shown in **Fig. 7.6**, where three examples are given to arrive at effective length of the pylon.

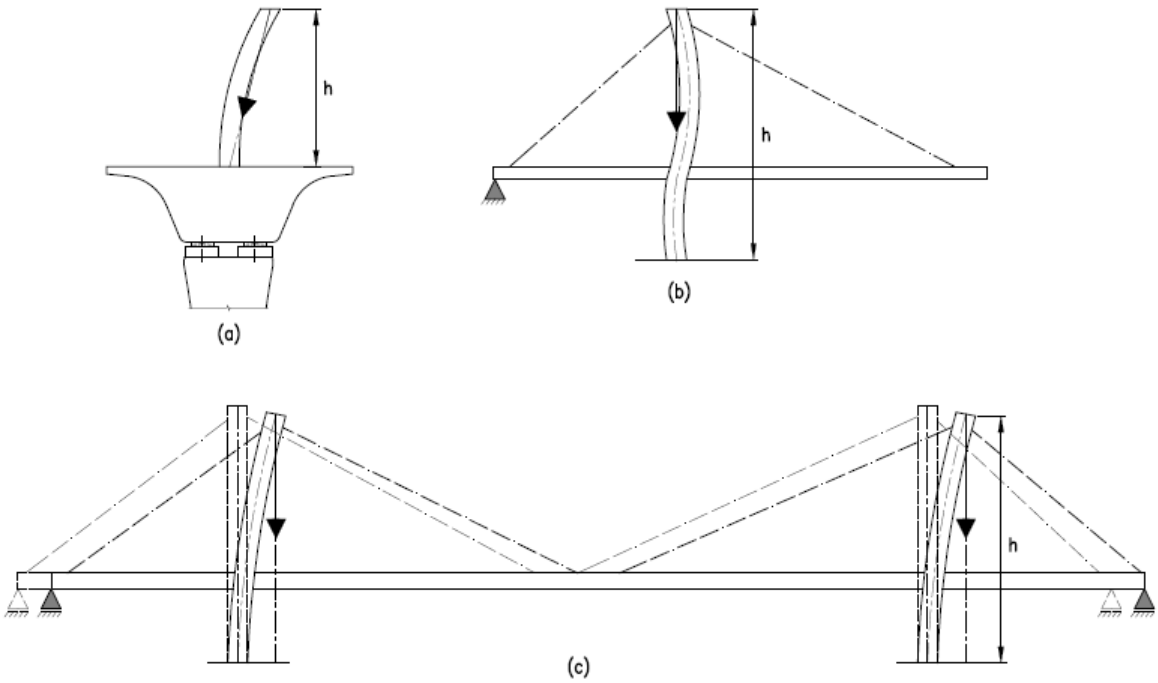


Fig. 7.6 Stability of Pylon

Fig. 7.6 (a) shows the lateral buckling of a free-standing pylon supporting a pure fan system at the top. The column effective length 1_c is equal to the pylon height 'h' due to the fact that the cable plane rotates with the anchor point at the pylon and the resultant of the stay cable forces still points towards the bridge axis. If this effect is neglected, then the effective length of the free-standing pylon would be twice the pylon height, i.e. a much more on the conservative side.

Fig. 7.6 (b) shows the buckling of a pylon in the longitudinal direction in a system where the girder is longitudinally supported at the end pier. In this case, the anchor cable leading from the fixed bearing to the pylon top restrains the pylon top in the longitudinal direction and the column effective length in the plane is therefore close to 0.7 times the pylon height.

Finally, **Fig. 7.6 (c)** shows the buckling of a pylon in a system where the girder has no longitudinal restraint through fixed bearings. In this case, the buckling is accompanied by a longitudinal sway of the girder. The column effective length is, consequently, twice the pylon height.

General method based on non-linear analysis as per IRC:112, can also be used to obtain second order effects in case of slender pylon.

When second-order calculations are performed, it is important that stiffnesses are accurately determined. It is therefore required that the analysis considers the effects of cracking, non-linear material properties and creep. This can be done either through a materially non-linear analysis or by using linear material properties based on a reduced secant stiffness as per IRC:112. Imperfections must be included as described in IRC:112 as these lead to additional first-order moments and consequently additional second-order moments in the presence of axial compression. Soil–structure interaction must also be considered.

7.5.4 *Method of Analysis for Specified Limit States*

Structural analysis shall be carried out by an appropriate model and analytical theory for specific limit states. The structural analysis shall be generally done with analytical models that can evaluate the stress produced by load acting on the structure. In addition, the analysis should consider the effects of creep and shrinkage.

a) Structural Analysis Method for Limit State of Serviceability

Calculation of member force to be utilized for examination under serviceability limit state shall be based on the Pseudo linear analysis as explained above. Usually, the stiffness to be utilized for calculation of member force may be estimated under the condition that the entire cross-section area is effective.

In regard to the estimation of displacements and deformations for structure, the concrete structure shall be generally regarded as an elastic body, and decrement of stiffness due to occurrence of cracks and effects of shrinkage and creep shall generally be taken into account.

b) Structural Analysis Method for Ultimate Limit State

In regard to the estimation of member force to be utilized for examinations of section failure under ultimate limit state, Pseudo linear analysis may be used in general. In case of the slender pylon or deck, results as per the above analysis may need to be verified with general method of second order effects based on nonlinear analysis, as per IRC:112.

For dynamic analysis, reference shall be made to **Chapter 9** of this guideline.

c) Structural Analysis Method for Fatigue Limit State

An estimation of member force to be utilized for examinations of fatigue limit state shall be based on the Pseudo linear analysis.

7.6 Initial Shape and Determination of Cable Forces

The cable stayed bridge consists of three principal components, namely deck/girders, towers and inclined cable stays. The deck system is supported elastically at points along its length by inclined cables so that it can span a much longer distance without intermediate piers. The dead load and traffic load on the deck system are transmitted to towers by inclined cables. High tension forces exist in cable stay which induce the compression forces in tower and parts of the deck system. Since high pretention force exists in inclined cables before live loads are applied the initial geometry and the prestress of cable stays are dependent on each other. They cannot be specified independently as for conventional steel and reinforced concrete bridges. Therefore, the initial shape, i.e., the geometric configuration and prestress distribution of cable stayed bridges, must be determined prior to analysing them.

In the design of these bridges, the assessment of initial cable forces and the procedure of stay stress adjustments during erection is very important, to respect the requested geometric profiles of deck and towers at the end of construction. At the end of erection, the girder longitudinal profile must satisfy aesthetic and functional requirements, possibly presenting a convenient pre-camber; the towers must keep the vertical profile, in order to avoid second order effects and to satisfy architectural demands.

Several techniques for the detection of initial stress distribution of pretension exist:

i) Simply Supported Beam Method

Assuming that each stayed cable supports approximately the weight of one segment, corresponding to the longitudinal distance between two stays, the cable forces can be estimated conveniently. It is necessary to take into account the application of other loads (wearing surface, curbs, fences, etc.). Also, the cable is placed in such a way that the new girder element is positioned correctly, with a view to having the required profile when construction is finished.

ii) Method of Continuous Beam on Rigid Supports

By assuming that under the dead load the main girder behaves like a continuous beam and the inclined stay cables provide rigid supports for the girder, the vertical component of the forces in stay cables are equal to the support reactions calculated on this basis. The tension in the anchorage cables make it possible to design the pylons in such a way that they are not subjected to large bending moments when the dead loads are applied.

iii) Optimization Method

In the optimization method of determining the stresses of the stay cables under permanent loads, the criteria (objective functions) are chosen so the material used in girders and pylons is minimized. When the internal forces, mainly the bending moments, are evenly distributed and small, the quantity of material reaches a minimum value. Also, the stresses in the structure and the deflections of the deck are limited to prescribed tolerances

iv) Shape iteration method

A finite element procedure for determining the initial shape of cable-stayed bridges under the action of dead loads and pretension in inclined cables is presented e.g., in Wang **[12]**. The cable-stayed bridges are analysed with all the geometrical nonlinearities with the available software. Based on a reference configuration and an assumed cable pretension force, the equilibrium configuration under dead load is then found. Further, by adjusting cable forces, a ‘shape iteration’ is carried out and a new equilibrium configuration, i.e., a more reasonable initial shape, can be determined. The shape iteration is then repeated until the desired tolerance is achieved.

7.7 Construction Stage Analysis and Adjustment of the Cable Forces

During the construction or service stage, many factors may induce errors in the cable forces and elevation of the girder, such as the operational errors in tensioning stays or the errors of elevation in laying forms. Further, the discrepancies of parameter values between design and reality such as the modules of elasticity, the mass density of concrete, the weight of girder segments may give rise to disagreements between the real structural response and the theoretical prediction. If the structure is not adjusted to reduce the errors during construction, they may accumulate, and the structure may deviate away from the intended design aim. Moreover, if the errors are greater than the allowable limits, they may give rise to the unfavourable effects to the structure. Through cable force adjustment, the construction errors can be eliminated or reduced to an allowable tolerance. Additionally in the service stage, because of concrete creep effects, cable force may need to be adjusted; thus, an optimal structural state can be reached or recovered.

For constructing the bridge and predicting the structural behaviour of the bridge during its erection procedure, the geometric configuration and forces in members of the bridge structure at each erection stage have to be known and examined in detail.

To reach the design aim, an effective and efficient simulation of the construction process step by step is very necessary. The objectives of the simulation analysis are

1. To determine the forces required in cable stays at each construction stage.
2. To set the precamber of the girder segment.
3. To find the consequent deformation of the structure at each construction stage.
4. To check the stresses in the girder and pylon sections

Construction stage analysis technique can be forward step analysis or backward step analysis

i) Forward Step Analysis

The forward process analysis of cable-stayed bridges during construction is performed by following the actual sequence of erection stages in bridge construction. Creep effects can be considered; however this method has limitation that actual lengths of the cables (which make the final elevation of the deck) are not known initially and hence requires some trials. But due to availability of the specialised commercial softwares to carry out this analysis trials and limitations of this method can be taken care.

ii) Backward Step Analysis

The backward process analysis of cable-stayed bridges during construction is carried out by following the direction reversed to the sequence of erection stages in the bridge construction. Backward step analysis, which starts with the final aim of the structural state and disassembles segment by segment in a reverse way. The disadvantage of this method is that the creep effects may not be able to be defined. However, values obtained from the assemblage process may be used in this analysis.

Section II – Design

7.8 Design of Cables

The design of cables for cable stayed must comply with requirements of this section. This section of the guideline is generally in line with **Fib 89 [1].**

7.8.1 *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

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

Where,

n is number of strand / wires

A_p Area of each strand / wire

f_{GUTS} Guaranteed ultimate tensile stress of the cable

7.8.2 *Ultimate Limit State for Cables*

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

$$F_{ed} \leq \frac{F_{GUTS}}{\gamma_m}$$

γ_m - 1.35,

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 they 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 into which the cable stay is connected, and, which transfer cable tension to the structure, shall be designed for Limit State condition as per the IRC and must be able to take force of 1.0 F_{GUTS} under ULS combinations.

7.8.3 *Seismic Design for Stay Cables*

All cables need to be designed for earthquakes, e.g., to avoid plastic deformations in the cable (values to be limited as per **Section 8.8.2**) considering flexural effects under large deformations of the structure.

Time history analysis as per chapter 9 should be performed to verify seismic demand and differential dynamic force conditions for any use of saddles in seismic zone IV and zone V. When seismic time-history analysis predicts net cable tension loads less than 10% of F_{GUTS}, cables and their anchorages shall be designed to assure adequate seismic performance (e.g. wedges are secured to cable in place). In addition, verification testing of cable lift off at the anchorage system and saddle shall be proposed by the cable supplier and approved by the designer on a project-by-project basis.

Saddles that depend solely on friction, if used in seismic zone IV and zone V, should be checked with detailed structural investigation of the bridge, such as time history analysis, using relevant site-specific ground motion input and friction properties of the saddle.

7.8.4 *Fatigue Limit State*

A) Stay Cable and Anchorages

The fatigue damage usually occurs at the anchorages, because of varying axial load and repeated bending caused by angular deviation at the ends. These angular deviations are principally due to cable stay vibrations caused by wind and due to rotation of the bridge deck.

- When using cable systems designed and tested for angular deviation during fatigue test for fatigue angular deviations and service angular deviations as per the recommendations in **Chapter 10**, fatigue angular deviation need not be considered in design.

- Damping of the cable vibrations by means of dampers, stabilizing stay cable textured stay pipe shall be done if required. In addition to Internal Dampers at stay anchorage's location, provision of external dampers shall be also explored, if cable vibration requirements are not met with.

- Hinging of the cable anchorage relative to the structure during the construction phase (for joint), or use of an effective procedure for adjusting the axial orientation of anchorages to avoid unforeseen permanent angular deviation between the cable stay and the deck or pylon can be adopted.

Further with above detailing care taken into consideration, the stay cables fatigue strength is guaranteed if following two conditions are met:

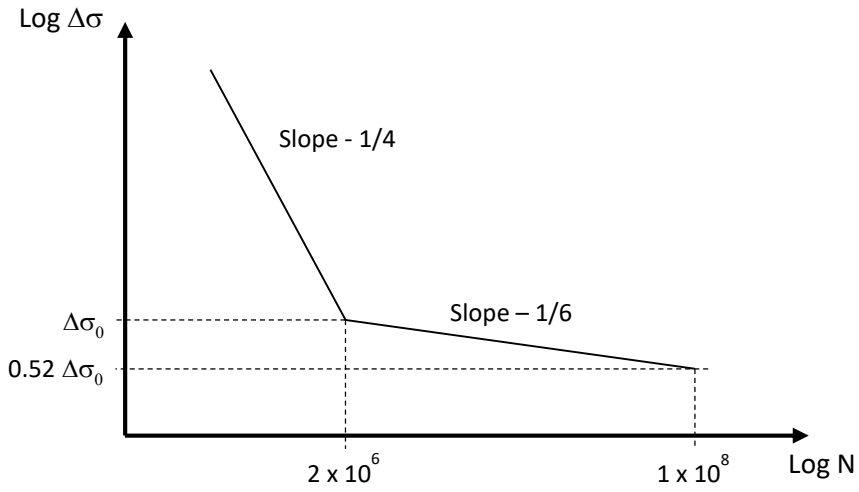
1) Qualification of cable system in accordance with **Chapter 10**

2) Verification of the cable system for variable stresses in cable (demand) compared to design strength of the fatigue stress range.

Annexure A2 of this guideline explains different approaches adopted internationally for fatigue design of stay cables. The fatigue design of stay cable to be considered for the cable stayed bridges as per the provisions mentioned below:

B) Design S-N fatigue P.C. Curve for Stay Cable

The shape of the S _ N Curve (Wohlers Curve) of a cable stay depends on the cable stay system. It has a bilinear shape and changes direction at 2 millions cycles for the stress range of $\Delta\sigma_0$



In the absence of additional data, it is acceptable to take the fatigue strength at 2 million cycles $\Delta\sigma_0$, of a cable stay being the same as the stress range $\Delta\sigma_{test}$ applied during the mechanical qualification test. **Chapter 10** recommends a $\Delta\sigma_{test}$ range of 200MPa for cable stayed bridges with upper stress of 0.45 f_{GUTS}.

As an approximation, the slope of the second part of the curve can be estimates to be 1/6. The endurance limit of the cable at 100 million cycles is therefore:

$$\Delta\sigma_0 \left(\frac{10^8}{2 \times 10^6}\right)^{-\frac{1}{6}} = 0.52 \times \Delta\sigma_0$$

C) Fatigue Verification

The cable stayed bridge for highway loading shall be designed to withstand 100 million crossings of the Fatigue Vehicle and it should therefore be checked that

$$\Delta\sigma_{fat} \leq \frac{0.52.\Delta\sigma_{test}}{\gamma_{Mf}}$$

$\Delta\sigma_{fat}$ – stress range due to fatigue vehicle as per IRC:6

γ_{Mf} – Partial Safety Factor – 1.25

The stress range $\Delta\sigma_{fat}$ due to fatigue vehicle shall be arrived at by calculating absolute difference in maximum and minimum stresses in cable due to passage of Fatigue Vehicle and multiplying by factor 1.4 to account for longer spans and uncertainty in load.

D) For other elements and details

Fatigue check for other elements of bridge for reinforcement, structural steel relevant codes of IRC shall be referred.

7.8.5 *Serviceability Limit State for Cables*

i) During Service

The Stay cables shall be checked for maximum stresses in rare combination.

f_{sis} = 0.5 f_{GUTS}

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

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

ii) SLS check under Construction and cable replacement

The permissible axial stresses during construction and cable replacement under rare SLS load combinations shall be as below

f_{sis} = 0.6 f_{GUTS}

CHAPTER - 8
SEISMIC EFFECTS

8.1 Introduction

Cable stayed bridges are long span and flexible structures, typically having long vibration periods, low damping, complex vibration modes, and usually subject to spatial variation of ground motion at different pylons/supports. The long fundamental period may result in low design seismic forces governed by the minimum design forces prescribed by **Clause 4.4** of IRC:SP:114. On the other hand, these bridges undergo large seismic deformations, needing consideration of second-order effects in their structural analysis and design. Supplemental energy dissipation may be useful in controlling the displacement. Due to coupling between the longitudinal, transverse, vertical and torsional modes of vibration, three-dimensional modeling and dynamic analysis considering adequate number of modes is necessary. The seismic design of these bridges shall be governed by IRC:SP:114. However, due to long spans and complex dynamic behavior, some additional considerations are required for seismic design of cable stayed bridges. The peculiarities of seismic behavior of cable stayed bridges and additional requirements for seismic design of these bridges, are discussed in this chapter.

8.2 Seismic Conceptual Design

The conceptual seismic design is an important step of the total design process. Seismic performance is dependent on the conceptual design, particularly the structural form of the cable stayed bridge, tower type, and cable layout. In the conceptual design phase, the structural form should be selected based on design considerations for static loads, seismic resistance and aerodynamic stability. To decide the structural form, cable layout and tower types, pylons and foundations, it may be desirable to carry out a parametric study amongst few chosen structural forms, tower types and cable layouts, to determine their impact on seismic behavior using the quick response spectrum method. Seismic conceptual design may require considerations of four design aspects which produce minimum seismic forces and seismic displacements: deck type, deck-tower connection, arrangement of side piers, and deck-side pier transverse connection. Reference may be made to **Chapter 4** and Calvi et. al. **[3]** for preliminary design recommendations. In general, for a cable stayed bridge with several auxiliary piers, it is beneficial for deformation and forces induced in the structure that all the side piers offer transverse restraint. Similarly, a full floating (without any
deck-pier connection) configuration with fuses and supplemental energy dissipation **Fig. 8.1** can result in reduced forces as well as controlled displacement. Under eccentric gravity loading and wind forces, the rigid fuses transfer the deck forces to the pylons, whereas under severe seismic loading the fuses yield, thus activating the dampers, resulting in energy dissipation.

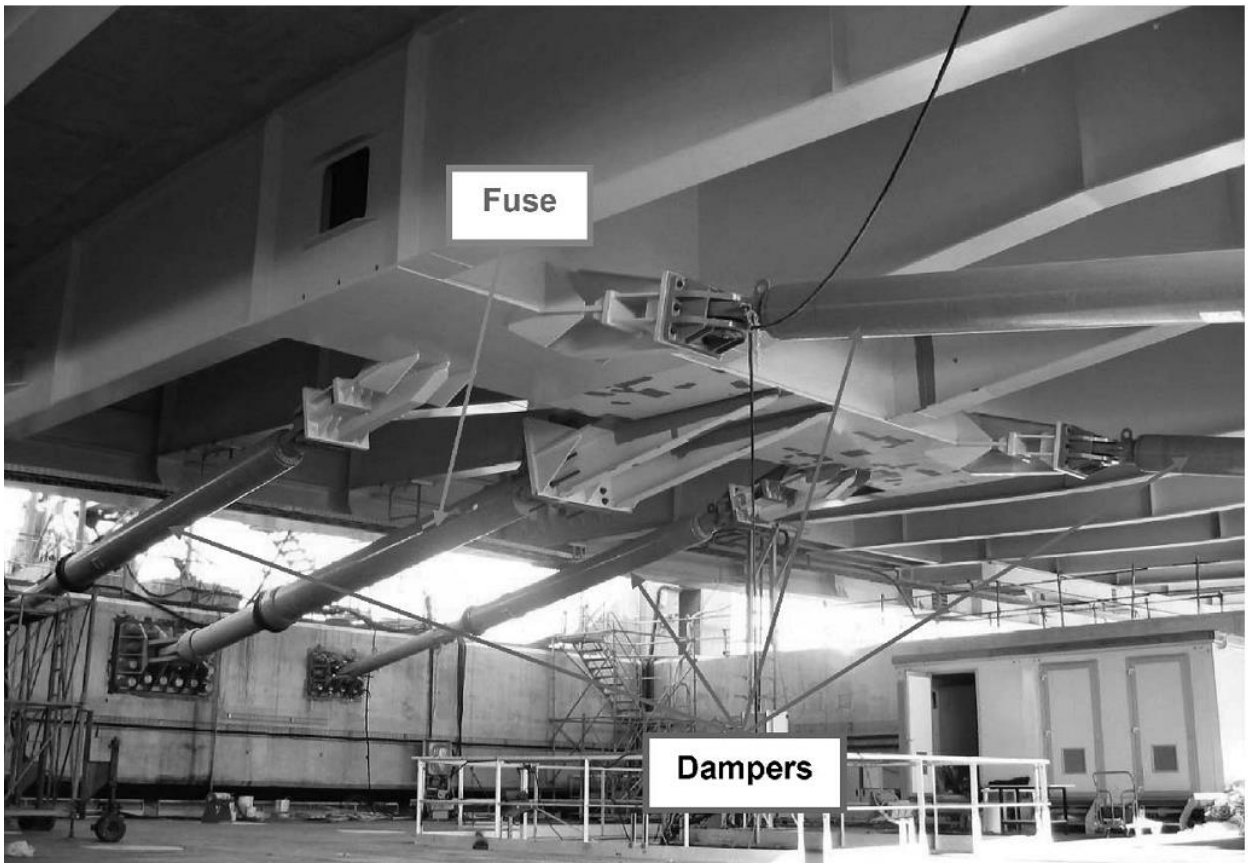


Fig. 8.1 Fuses and Supplemental Damping used for Fully Floating Superstructure of a Cable Stayed bridge [1]

8.3 Seismic Considerations

Cable stayed bridges are composed of various types of components with different stiffness and damping characteristics. These result in coupled modes and hence require some special considerations during their seismic design. Following are the main points to be considered in seismic design of cable stayed bridges:

i) These bridges usually do not have clearly distinguishable modes of vibrations in longitudinal, transverse, and vertical direction, as in the case of simple bridges and buildings. Due to coupling of modes, seismic action in one direction causes forces in other direction. The pylons are subjected to axial force and bi-directional bending actions due to longitudinal, transverse and vertical components of earthquake, along with gravity loads. Therefore, the cable stayed bridges should be designed for combined effect of the two horizontal and vertical components of earthquake. The 100:30:30 combination rule **(Clause 3.2.2 of IRC:SP:114)** can be used for combining the member actions obtained from response spectrum analysis. In case of time history analysis, all the three components of ground motion need to be considered and the effect is automatically combined in appropriate proportion.

ii) The deck-pier connection and expansion joints may get damaged when the movements of bridge component exceed their capacity (refer **Chapter 11**); these 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. Use of dampers and fuses may be considered for reducing the excessive movement.

iii) Long span cable stayed 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 because 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 given in **Section 8.5**. The consideration of spatial variation is even more important in case of bridges having piers/towers located on different types of soil strata.

iv) Cable structures depict geometric nonlinearity. Due to sag in the cable, it shows a nonlinear force-displacement behavior, as part of the applied force causes straightening of cable, without any strain. **Chapter 7** provides different methods to take into account the geometric nonlinearity, including that due to cable sagging. Further, the cables cannot take any compressive force and therefore, it should be ensured that the cables are in tension during the whole time-history of the earthquake. The pre-tensioned cables of bridges may come in compression due to vertical vibrations of the deck caused by combined effect of horizontal and vertical ground motions; their status of tension should be checked under seismic conditions.

v) Long cables may have their own period of vibration close to the period of vibration of the pylon or superstructure. This may lead to swapping of energy between these components. This phenomenon is called cable-bridge interaction and must be considered in seismic analysis of cable stayed bridges. This can be easily considered by dividing the cables into smaller elements and lumping the masses at intermediate nodes.

vi) The long spans and tall pylons cause large overturning moments at the base requiring large size foundations. Also, the foundation of the bridge may destabilize due to soil liquefaction. The foundations shall be modelled considering the nonlinear behavior of soil as explained in **Section 8.4.5**. Further, the foundation should be designed for capacity forces being transmitted by the pylon, including overstrength, according to **Clause. 6.8 of IRC:SP:114**.

vii) High strength cables used to support the cable-stayed superstructure have very low ductility and should be designed to remain elastic, even during the most severe earthquake.

8.4 Modelling and Analysis for Seismic Design

8.4.1 *3D Modelling of Structure*

Cable stayed bridges undergo complex and coupled modes of vibration under earthquakes. The torsion of superstructure is combined with transverse vibrations. Similarly, the pylons are also subjected to torsion as a result of horizontal and vertical vibrations of the superstructure. Therefore, it is important that a 3-D model is used for seismic analysis and all the three components of earthquake motion are considered to act simultaneously. A single pier or single spine model of superstructure is not adequate to consider the torsional effects. The 3D model of a cable stayed bridge can be developed using line or frame elements or multiple types elements, as per **Section 7.2** of these guidelines. If a single spine model of a cable stayed bridge having a single plane of cables along the centerline of the super-structure, is used, the torsional inertia of the superstructure and spatial location of the bearings with respect to centerline of the superstructure need to be modelled adequately. This can be done by using rigid appendages projecting from the spine and connecting the bearings with the spine.

8.4.2 *Adequate Mode Participation*

The dynamic characteristics (i.e. free vibration modes) of a structure are classified according to their action direction: longitudinal, transverse, vertical and torsional. The natural vibration modes can occur in each of these directions and there can be coupling between the modes. The vibration modes of cable stayed bridge are complex. The initial modes of vibration show long periods, of the order of several seconds, and these are fundamentally deck modes. These are followed by cable vibration modes, coupled with the deck. The tower vibrations are usually higher-order vibration modes, which can be coupled with the deck depending on support conditions. The modes are difficult to separate when they are highly coupled. For typical cable stayed bridges, there are strongly coupled modes like bending and torsion in the three orthogonal directions. The vertical vibrations of deck, induced due to vertical ground motions at the tower supports, could be quite important; so also, the torsional motion of the bridge induced due to two simultaneous components of horizontal earthquake motions at tower supports.

As the seismic response in case of cable stayed bridges, is contributed by a large number of coupled modes, it is necessary to perform dynamic analysis for seismic design of cable stayed bridges. The equivalent static method which is essentially a single mode method is not adequate for these bridges. The dynamic analysis can be performed either by modal analysis method using response spectrum, or a modal response history analysis using appropriate ground motions, or a direct time history analysis. Naturally, the direct time history analysis is the most accurate being a rigorous analysis procedure, which inherently considers the contribution of all the modes of vibration. However, this may be a time-consuming method and requires skill in selection of appropriate ground motions. Therefore, it is recommended to use a modal analysis for preliminary design of cable stayed bridges, and the direct time history analysis for design verification at final stage of seismic design.

While using the mode superposition method, it is important that enough modes are considered to result in 90% mass participation in all the directions. Sometimes it is difficult to get 90% mass participation in each direction of the cable stayed bridge. In such a case,

Ritz vectors may be tried, otherwise it may be required to perform a direct time history analysis, considering three components of ground motion.

8.4.3 *Design Hazard Level, Importance Factor and Response Reduction Factor*

Euro code suggests that “for bridges where the seismic response may be dominated by higher mode effects (e.g. cable-stayed bridges), or where the detailing of plastic hinges for ductility may not be reliable (e.g. due to a high axial force or a low shear-span ratio), a behaviour factor of q=1 is recommended, corresponding to elastic behaviour.” Accordingly, the cable stayed bridges shall be designed for the DBE level earthquake forces with response reduction factor, R equal to 1.0 and the importance factor, I =1.5. The earthquake action in both the horizontal and vertical directions shall be considered.

8.4.4 *Damping*

The damping in Cable Stayed Bridges is characteristically low, smaller than 5% of critical damping, normally assumed in the bridges **[2,3,4].** The total damping in the bridge depends on the damping in individual components of the bridge; deck, towers, cable system, pylons, foundation, and founding soil. The damping in each of the component members is different. The dissipation mechanism in the seismic response of cable stayed bridge comprises of structural damping, bearing friction, internal slip of wires inside the cables (friction), foundation radiation, aerodynamic damping and system damping, due to interaction between the deck, cable system and the tower. To carry out seismic analysis, the equivalent damping can be worked out based on weighted damping proportional to the strain energy stored in each component. The estimated value of damping is also sensitive to the relative participation of modes. The damping reduces for higher modes and with increasing span. The damping matrix required for direct step by step integration method can be formed using Rayleigh damping. This requires specification of the damping in two modes of the vibration. The damping in Active or Passive devices can be included in the modeling of the device in the mathematical model in the form of hysteretic damping.

In a simplified case, the material of the deck can be used to decide the damping factor. In absence of a better estimate of damping, 2%, 3%, and 4% of critical damping can be considered for cable stayed bridges with steel, steel-concrete composite and RC decks, respectively. For hybrid decks, the lowest value applicable to main span will be used. For construction stage pylon material to be used to select the appropriate amount of damping. In direct time history analysis, the 2%, 3%, and 4% Rayleigh damping may be considered corresponding to fundamental mode and the mode resulting in 90% mass participation.

8.4.5 *Consideration of Effects of Soil*

In general, soil below the founding level has a complex interaction with the structures during earthquake. To consider the Complete soil-structure interaction, soil is modelled as continuum and seismic wave propagation through the soil is also considered. The soil flexibility, wave propagation, kinematic interaction, inertial interaction, and radiation damping, all the effects of SSI are considered together in this approach. This is the most accurate method of assessing SSI, but computationally intensive. Further, this requires extensive field and lab investigation of the founding soil, preferably up to the bed rock level. The soil properties (stiffness, strength and hysteresis parameters) estimated through the

investigations are required for nonlinear continuum material modelling of the soil. Further, the free field ground motions selected at the surface level should be de-convoluted at level of the base of the FE model considered. As soil is a semi-infinite medium and only a finite domain is considered in the finite element model, free-field or absorbing boundary conditions are required to be applied at the domain boundaries. Many of the available computer software provide these modelling options.

In the soil-structure interaction, two effects of soil are the most important: (i) soil amplification of ground motion due to impedance contrast in the medium of wave propagation, and (ii) soil flexibility resulting in elongation of period and permanent deformation of the structure. In the decoupled method of SSI, these two effects are considered independently. In this method, the soil may be modelled using nonlinear (bi-linear or multilinear) springs and the wave propagation through the soil is considered separately, using 1D wave propagation. Several public domain software (e.g. SHAKE and DEEPSOIL) are available for 1D wave propagation. These can consider the soil as equivalent linear or nonlinear layered medium. The soil properties for each layer need to be specified. The de-convoluted ground motion is applied at the base of the soil column and the seismic motion at different levels in soil is obtained using the 1-D wave propagation. The ground motion obtained at different levels in the soil is applied at the corresponding springs. It is a reasonably accurate and efficient method, most commonly used internationally for practical SSI problems. The American Petroleum Institute (API) and other standards recommend this procedure and provide guidelines for developing nonlinear/bilinear soil-spring properties. Based on API and other guidelines a separate document, IRC B-3.7 **Annexure-F [5],** has been developed by IRC for estimating the modelling parameters of soil springs in different types of soils, including liquefiable soils. This document can be used to develop the soil spring models for pile and well foundations. Shallow foundations are less probable in case of cable stayed bridges, but such equivalent spring models are available in literature for shallow foundations also.

A further simplified procedure to consider the major two effects of SSI is possible, in which the soil amplification is considered using the code-based response spectra/amplification factors or using the site-specific earthquake design parameters, whereas the soil flexibility effects are considered using nonlinear/bilinear equivalent springs. However, a nonlinear time-history analysis is needed in this case also, which can be handled by most of the software available and used in Indian industry now-a-days.

For quick analysis for preliminary design or if the designer does not want to go for the nonlinear analysis there is an approximate method to use this modelling in linear analysis software, by performing iterations. In this iterative equivalent linear analysis, first a probable displacement of foundation is estimated using the fixed-base design forces or any other appropriate assumption. The secant stiffness corresponding to estimated displacement is computed using the bilinear force-displacement curves for soil spring. Then, using the secant stiffness as the equivalent linear stiffness, analysis performed using any of the commonly used software, and the resultant foundation displacement is computed. If it is close to the originally assumed displacement, the solution is accepted otherwise further iterations are performed using a new assumed foundation displacement. The advantage of this iterative equivalent linear procedure is that it can be used with response spectrum method also and provides reasonably accurate results for practical design. However, in

case of important cable stayed bridges, it is desirable to make a final verification of the design using nonlinear time history analysis with bilinear soil springs or with complete SSI modelling.

8.5 Spatial Variability of Ground Motion

Spatial variability of the ground motion is a well-known phenomenon which may lead to multiple support excitations leading to different ground motion time histories occurring at various supports of the structure. In normal buildings, structures and ordinary bridges, identical support motion is assumed; spatial effects of ground motion are not considered important because of smaller lengths of structures. The equation of motion for multi-support excitation is well known; now available in standard text books on Theory of Vibration or Dynamics of Structures and research papers. The solution of these equations of motion in the time domain leads to the determination of the structural response in earthquakes for this type of motion. The most important aspect is to define time history of multi-support excitation at different support points of the structure using existing methods. The spatial variability of ground motion is important only for long spans. Spatial variability effects involve three main mechanisms: wave passage effect, incoherency effect, and site-response effect. Wave propagation effect represents the time of travel by seismic waves from one support to another. However, when the wave travels in ground the coherency (correlation between the ground motions at the two supports) of waves is lost with travel distance due to multiple reflections and refractions in the propagating medium. The coherency is further lost during propagation through the soil strata below the foundation up to bedrock. Out of these, the site amplification effects, i.e. effect of variation in the soil strata below the different foundations is considered to be most significant **[6].**

Consideration of spatial variation of ground motion is a crucial issue in seismic analysis of long span bridge. **Clause 4.5** (**Table 4.3**) IRC:SP:114 recommends a dynamic analysis with spatial variability of ground motion in case of continuous bridges of length between expansion joints > 150 m. The limit of 150 m length can be interpreted as the distance between the farthest supports (piers/abutments) interconnected by a continuous superstructure. The Eurocode (EC8, Part-2) **[6]** provides two conditions for consideration of variability in ground motion:

(i) The variation of soil conditions at the two supports is such that it can be classified into different site classes (Class A to E defined in **Table 2** of IRC B-3.7 **Annexure-F [5]),**

OR

(ii) The distance between the farthest supports of a continuous deck is greater than *Lg*/1.5, where *Lg* depends on the site class, (which in turn is defined in terms of shear wave velocity) given as:

Site Class	A, B	C	D	E
L_g (m)	600	500	400	300

Note: Notwithstanding the above table, as specified in the preface, these guidelines are applicable only for bridges having main span up to 500 m.

It is possible to consider the ground motion variability using time history as well as response spectrum method. In time history analysis (linear or nonlinear) of a cable stayed bridge, the ground motions used in seismic analysis should take into account the effect of wave propagation, loss of coherency with distance, and variation in soil conditions below the different supports. The correlated ground motions at different supports of a cable stayed bridge can be obtained using the procedure described in **Annexure-D** of EC8, Part-2. A simple but approximate approach to consider the wave propagation effect and the ground response effect is by introducing a time lag in the ground motions at bed-rock level and then propagating the bed-rock motions through the soil strata below the corresponding supports to account for the variability is soil conditions. The time lag may be estimated as the travel time for the shear wave for the spacing between the supports and the effective shear wave velocity in the bedrock, whereas 1D nonlinear or equivalent linear wave propagation may be used to consider the effect of soil strata below different supports.

In response spectrum based seismic analysis of a cable stayed bridge, a Multi-Support Response Spectrum (MSRS) method **[7]** can be used to consider the spatial variability of ground motion. The MSRS rule evaluates the mean peak response in terms of the response spectra and mean peak ground displacements at the support points of the structure, and the coherency function that characterizes the spatial variability of the support motions. The MSRS method has been adopted by EC8 (Part-2) **[6].**

The EC8 (Part-2) also provides a simplified Pseudo-Static Method. In this method, the inertia response is accounted for by using a single input seismic action for the entire structure (e.g. a single response spectrum or corresponding time history sets), corresponding to the most severe ground type underneath the different bridge supports. The spatial variation of the seismic action is estimated by pseudo-static effects of appropriate displacement sets, imposed at the foundation of the supports of the bridge deck. The following two sets of horizontal displacements (**9.5.1 and 9.5.2**) are applied separately, in each horizontal direction of the analysis, on the relevant support foundations or on the soil end of the relevant spring representing the soil stiffness. The effects of the two sets need not be combined.

8.5.1 *Set A Displacements*

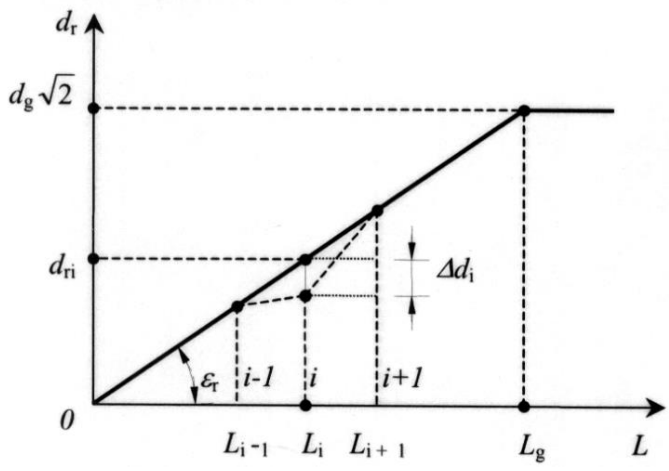


Fig. 8.2 Displacement Set A

Set A consists of relative displacements:

$d_{ri} = \varepsilon_r L_i \leq d_g\sqrt{2}$ where $\varepsilon_r = d_g\sqrt{2}/L_g$

applied simultaneously with the same sign (+ or –) to all supports of the bridge (1 to *n*) in the horizontal direction considered (**Fig. 8.2**).

where;

d_g is the design ground displacement corresponding to the zone factor ground type of support *i*, given as 0.57Z, 0.77Z, and 0.95Z (in m), for Soil Types I, II, and III, respectively.

L_i is the distance (projection on the horizontal plane) of support *i* from a reference support *i* = 0, that may be conveniently selected at one of the end supports;

L_g is the distance beyond which the supports are not interconnected by a continuous superstructure and the ground motions may be considered as completely uncorrelated.

8.5.2 *Set B Displacements*

Set B covers the influence of ground displacements occurring in opposite directions at adjacent piers. This is accounted for by assuming displacements Δd_i of any intermediate support *i* (>1) relative to its adjacent supports *i*-1 and *i*+1 considered un-displaced (**Fig. 8.3**).

$\Delta d_i = \pm\beta_r\varepsilon_r L_{av,i}$

where:

$L_{av,i}$ is the average of the distances $L_{i\text{-}1,i}$ and $L_{i,i+1}$ of intermediate support *i* to its adjacent supports *i*-1 and *i*+1 respectively. For the end supports (0 and *n*) $L_{av,0} = L_{01}$ and $L_{av,n} = L_{n\text{-}1,n}$;

β_r is a factor accounting for the magnitude of ground displacements occurring in opposite direction at adjacent supports. β_r = 0.5 when all three supports have the same ground type; β_r = 1.0 when the ground type at one of the supports is different than at the other two.

ε_r is as defined for set A above. If a change of ground type appears between two supports, the maximum value of ε_r should be used.

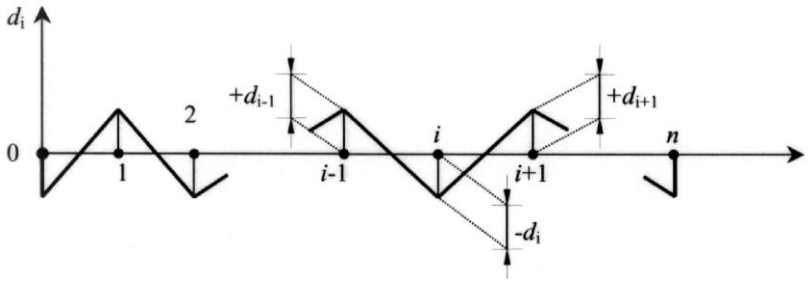


Fig. 8.3 Displacement Set B

Set B consists of the following configuration of imposed absolute displacements with opposed sign at adjacent supports *i* and *i*+1, for *i* = 0 to *n*-1 (as shown in **Fig. 8.3**).

$d_i = \pm\Delta d_i/2$; $d_{i+1} = \pm\Delta d_{i+1}/2$

In each horizontal direction, the most severe effects resulting from the pseudo-static analyses for support displacements of 9.5.1 and 9.5.2 shall be combined with the relevant effects of the inertia response obtained from response spectrum or time history analysis, by using the SSRS rule (square root of the sum of squares). The result of this combination constitutes the effects of the analysis in the direction considered. For the combination of the effects of the different components of seismic action, the 100:30:30 as per **Clause** 3**.2.2** of IRC:SP:114 shall be used.

8.6 Seismic Protection Devices

Besides the traditional seismic design for strength and ductility, the incorporation of certain passive and active devices for vibration control in cable stayed bridges can be quite effective in enhancing its ability to resist earthquakes and mitigation of cable vibrations. The seismic protection devices are useful in improvement of seismic behavior and controlling seismic displacements. The Seismic Protection Device basically works on the principle of dissipation of energy; thus, these devices reduce the structural response. Though several types of Seismic Protection Devices are available: Passive, Active, Semi-Active or Hybrid for vibration control; it is necessary to define goal of introducing these devices prior to their use for seismic protection. Some of the goals that can be considered in Cable Stayed Bridges are following: achieve elastic behavior of the bridge for a more frequently occurring earthquake, to improve seismic behavior in a large earthquake, to minimize deck displacements in earthquake, to control vibrations in wind, small earthquakes and traffic loads. The deck displacement without any seismic protection device is usually very high. It is often required to reduce these displacements. In other case the designer will have to accommodate these movements in large expansion joints. The design of seismic protection device and joint design where these devices are introduced should be done together in these bridges. The common passive devices that are recommended for cable stayed bridges are: Viscous damper, Hysteretic damper, Friction damper and Tuned Mass Damper.

The choice of a seismic protection device is guided by several variables. In fact, the design of a device should be an integrated process that should include the force transmitted to the structure, displacements in other directions, devices cost and their installation cost, joints cost and maintenance cost of everything involved during the life cycle of a cable stayed bridge.

One of the key factors in the dynamic behavior of a cable stayed bridge is the connection between the deck and the tower. The current trend in the design of cable stayed bridge is to incorporate seismic devices in the deck tower connection. The objective is to control seismic displacements and the seismic force and to help maintain the towers in elastic range during the earthquake.

CHAPTER - 9

WIND EFFECTS AND AERODYNAMIC DESIGN CONSIDERATIONS

9.1 Introduction

In this Chapter an effort has been made to guide the reader in understanding the basics of wind and its effect on a bridge as it interacts with the wind. Bridges of moderate spans can be designed for this effect by dealing with the problem as a quasi-static one. For longer spans, essentially the aerodynamic approach has to be pursued. Cable-stayed bridges are generally in a span range so as to fall in the latter category. The various kinds of related phenomena that can occur in a cable-stayed bridge are briefly described. Also given are approximate/ empirical relations for assessing the responses of a bridge, besides information and guidelines for its design. It must be emphasized, however, that while based upon the aforesaid information, a preliminary design can be prepared, for the wind-sensitive bridges, the design will require to be proved through further investigation. It will be prudent to include an appropriate period (say, about 6 months) for this purpose in the time-lines of the project.

9.2 Wind Characteristics

Wind flow generation is a result of conducive meteorological conditions, primarily pressure differentials, and manifests itself into a variety of patterns. These are monsoon winds, localized dust storms, thunderstorms, cyclones and tornados, all of which are relevant in the Indian context In the Indian context, except Tornados which are an infrequent phenomenon in India. The east coast experiences greater cyclonic activity compared to the west coast. In cyclones, high-speed winds are accompanied by heavy rains and flooding. Large-scale tropical storms formed over South Pacific and the Indian Ocean are called Cyclones, while they are called Hurricanes when formed over the North Atlantic Ocean and Northeast Pacific, and they are called Typhoons when formed over the Northwest Pacific Ocean.

A trace of wind velocity at any point in space can be generally depicted as shown in **Fig.9.1** It is obvious that wind is a random time-variant phenomenon, which can be interpreted as the sum of (i) a time-constant, spatially variable mean wind speed, and (ii) superimposed, time and space variable turbulence part. Whereas this is representative of various storm types, the actual structure would vary amongst them. For example, both cyclones and tornados, **Fig.9.2** are characterized by swirling winds of very high speed, reaching 300 km/hr. Cyclones are a broadband phenomenon covering hundreds of kilometers, with the major wind velocity content being tangential. Essentially, structures experiencing cyclones will be subjected to similar mean wind profiles like pressure system winds, but the associated turbulence will be higher. The main difference of this storm compared to a pressure system wind is, of course, the large wind speed blowing for several hours but from different directions caused by the rotational component of the storm. These emanate from the sea, and, cover vast areas of land near the coasts. Tornadoes, on the other hand, are a narrow band phenomenon, covering no more than a few hundred kilometers transversely. These are essentially an in-land activity, covering comparatively small areas. While a tornado generates very high tangential wind velocity, it is also accompanied by substantially potent updraft and radial velocities. Considering the rare occurrence of tornadoes in India, though the

associated speeds are high, these speeds are associated with a very high return period. The design cannot be economically viable considering such rare occurrences, and hence shall not be considered in the design for bridges. Hence, for the design, consider the speeds in the IRC:6 **[1]** which is in line with IS:875 (Part 3) [2] covering more frequent monsoons and cyclones.

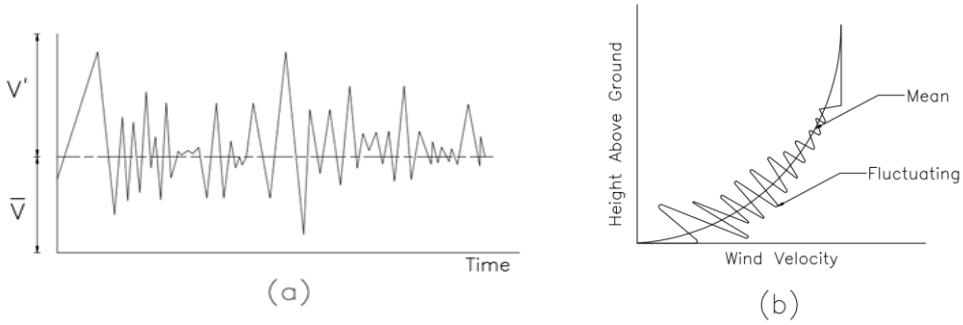


Fig 9.1 Depiction of Wind Velocity in Time – Space Dimension

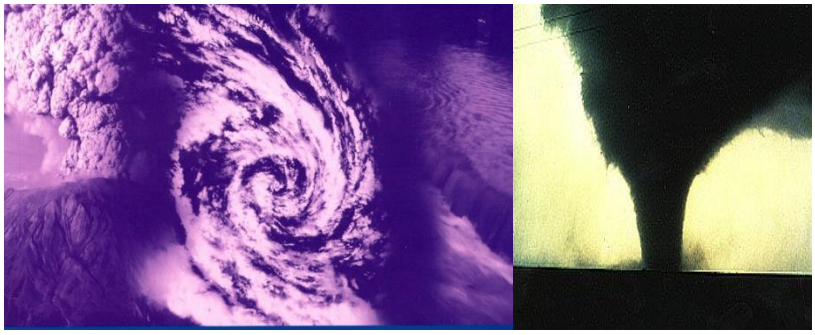


Fig. 9.2 Cyclones and Tornadoes

The wind being an inherently dynamic phenomenon, all structures will undergo a dynamic response to a smaller or greater degree, when these obstruct its flow. Structures that are comparatively stiff (say, with natural frequencies greater than 1 Hz), will respond in a quasi-static manner, and can be dealt with accordingly. The quasi-static approach given in IRC:6 **[1],** apparently addresses shorter span bridges. Longer spans and more flexible ones, such as cable-stayed bridges, which will fall into the wind–sensitive category, are expected to experience a predominantly aerodynamic response.

9.3 Dynamic Characteristics

The dynamic characteristics of cable-stayed bridges for wind-induced oscillations invoke great interest. These, being flexible structures, have low natural frequencies, generally less than 1Hz, and, are consequently more prone to vibrations. The vibration of cable-stayed bridges may involve a single member (deck, cable or tower) or the entire structure. Cable-stayed bridges tend to have very closely spaced natural frequencies and often significant structural coupling of flexural and torsional modes of vibration due to separation of mass center and stiffness center of the deck.

The frequency of vertical oscillation of bridge deck depends on the stiffness of stay cables in the vertical direction, stiffness of the girders, and mass of the structure. The stiffness mainly depends on main span length, the ratio of side span to main span length, the ratio of pylon height above bridge deck level to main span length, and the cross-sectional area of stay cables. The torsional frequency of the bridge deck depends on its torsional stiffness, e.g. a box girder is stiff while an “open’’ cross-section is weaker in torsion. Towers can also affect the torsional behavior of the deck. However, the tower is effective in torsion only if the tower shafts are tied with cross beams. A-shaped pylons with inclined cables provide higher resistance to torsional oscillations of the deck than those with an H-shape, with double plane cable arrangement (**Fig. 3.2** of these guidelines), though the latter have been frequently deployed in bridges of up to 500 m span.

Cables arranged in a single plane and connected to the deck along the centerline provide no resistance to the torsional rigidity of the deck. Such bridge decks have low torsional natural frequencies and low torsional damping. On the other hand, cables arranged in two planes and connected to the deck along its two edges provide a high degree of resistance to torsional oscillations. The resistance to lateral bending mode is mainly offered by the deck. The longitudinal oscillation of the deck is governed by the resistance offered mainly by the tower where bearings in the vertical plane can be provided to restrain transverse movement without impeding longitudinal movements.

Individual cables being flexible, possess very low structural damping, with a very much lower mass in comparison to deck or pylon. These often get excited in the vertical plane.

The vibration characteristics of a cable-stayed bridge, after completion, are quite different from that of the bridge during construction. The various procedures for erection, which include quite commonly the cantilever mode, are discussed in **Chapter 5, Section 4.2.3** of these guidelines.

During the various erection stages, mass (including the construction equipment) and stiffness are changing drastically, which affects the vibration characteristics and hence the corresponding wind responses. Therefore, often the bridge may be more vulnerable during the construction stages, in comparison to the completed bridge.

9.4 Wind Effects

9.4.1 *Dynamic Wind Effects on Bridge Deck*

The major wind induced problem in cable bridges is on account of the oscillations in the deck. These are classified broadly as limited amplitude oscillations and divergent amplitude oscillations **Fig. 9.3**. Prolonged limited amplitude oscillations could cause unacceptable stresses and fatigue damage in bridge components and discomfort to the users, whereas divergent amplitude oscillations may lead to distress or even failure.

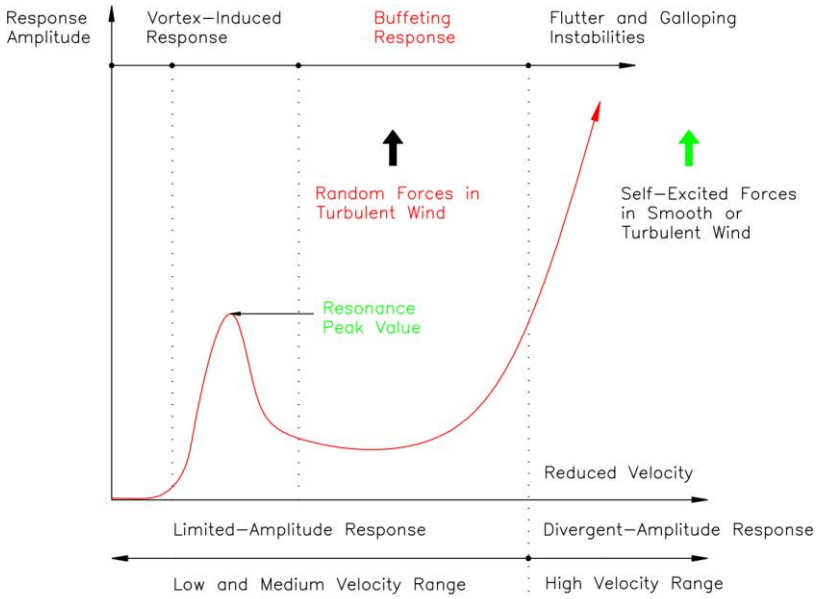


Fig. 9.3 Various Forms of Aerodynamic Response Exhibited by the Deck in Different Wind Velocity Ranges (Wind Velocity Range indicates Reduced velocity, Vre = V/fB, where f is frequency of vibration and B is a typical dimension)

9.4.1.1 Limited Amplitude Oscillations

These can be caused either by vortex induced oscillation or buffeting.

Vortex Induced Oscillation: These oscillations are caused due to the shedding of vortices alternatively from the top and bottom surfaces of the deck. The vortices produce periodic cross wind forces which cause oscillations either in vertical or torsional modes. The frequency of shedding of complementary pairs of vortices is expressed in non-dimensional form by the Strouhal number 'S', (S= fD/V) which is typical for any particular geometrical shape, where *f* is the vortex shedding frequency, *D* is the reference dimension of cross-section and *V* is the wind speed. When the frequency of vortex shedding, f, coincides with a natural frequency of the bridge deck, large amplitude oscillations may occur. This type of oscillation may occur over one or more narrow ranges of wind speeds.

Buffeting: The wind in the vicinity of earth's surface is turbulent. Turbulence causes buffeting and leads to fluctuating forces and moments on the bridge components. This action of the turbulent wind is experienced over a wide range of frequencies. The bridge structure must be analyzed for such buffeting responses.

9.4.1.2 Divergent Amplitude Oscillations

In divergent amplitude oscillations, the amplitude can continue growing until possible collapse. This is a consequence of loss of damping and/or stiffness arising from interactions of aerodynamic forces with changing deformations of the deck. These oscillations mainly manifest in the form of galloping and flutter due to instabilities in the vertical, lateral and

torsional motions either in isolation or in combination. Single degree of flutter is also called galloping.

The phenomenon of flutter arises in the form of single degree of freedom motion or coupled motion. Classical flutter arises due to the coupling of torsional and vertical motions. Flutter may ultimately lead to collapse unless it is restricted by the nonlinearity of the structure. Collapse of the Tacoma Narrows suspension bridge in the year 1940 was a landmark failure on account of flutter. The critical speed for flutter of a deck section can be arrived at by model testing in the wind tunnel. Alternatively, by complex eigenvalue analysis of the bridge the theoretical critical wind speed for flutter can be estimated. Theoretical analysis requires motional derivatives (flutter derivatives) for the deck cross-section which are again obtained from wind tunnel testing.

The cause and effects of wind induced vibrations are listed in **Table 9.1** [after ref. 3]

Table 9.1. Wind–Induced Vibrations

Types of Vibration	Turbulence Induced (buffeting)	Vortex Induced	Motion Induced (Flutter)
Cause	Random fluctuations of wind speeds in atmospheric boundary layer	Regular shedding of vortices from the body	Rhythmic motion of body in the flow
Excitation Forces	Along and transverse to wind direction	Mainly transverse to the wind direction	Mainly transverse and torsional
Effect	Dynamically amplified internal forces	Sharp resonance	Self - excitation instability

9.4.2 *Dynamic Wind Effects on Tower*

The dynamic wind effects on tower could be in the nature of along wind oscillations and on account of vortex induced oscillation (limited amplitude). Galloping is possible but is not expected to be commonplace. It has been found that the rectangular sections with width to depth ratio less than five are prone to galloping. Further, it may be important to consider galloping in the design of towers during erection before the cables and decks are installed.

9.4.3 *Stay Cable vibration under Wind*

Stay cable strands with a circular cross section, can oscillate at certain velocity of wind. This is called the ‘singing’ of the cable. Likewise, when provided as a group of 2 or 4 ropes or strands in place of a single rope or strand, in order to use smaller sizes, there is the possibility of wake vibration in the leeward element, **Fig. 9.4**

Another form of wind induced vibration that can occur in the stays when rivulets flow down, **Fig. 9.5** the stay due to the falling of rain and there is simultaneously a flow of wind. This is called rain-wind vibration, and, is a greater possibility in longer cables.

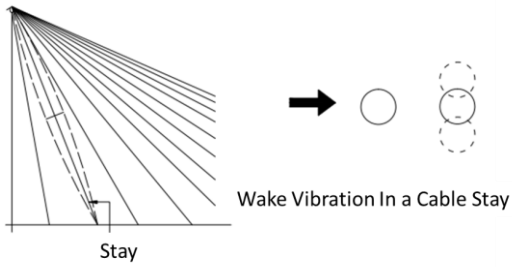


Fig. 9.4 Wake Induced Vibration of a Stay Cable

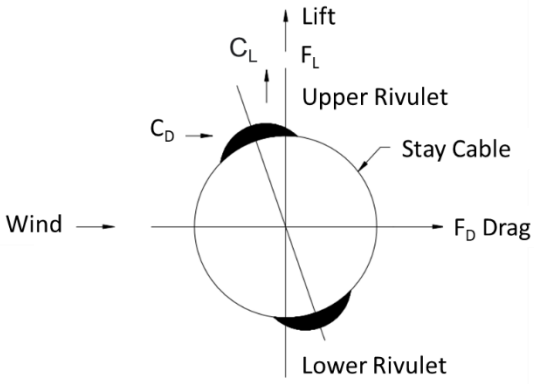


Fig. 9.5 Rain-Wind Vibration- Formation of Rivulet

Therefore, wind actions that shall be considered for the design of cable stayed bridges, besides the direct or quasi-static wind load, are as follows:

(i) Flutter

(ii) Galloping

(iii) Vortex induced oscillation

(iv) Buffeting response

The wind actions may produce the following effects:

(i) Excessive forces, stresses or deformation of the bridge or its elements

(ii) Repeated dynamic stress or deformation causing fatigue of bridge or its elements

(iii) Aerodynamic instability

(iv) Excessive dynamic movement causing discomfort to the users.

9.5 Principles for Wind Resistant Design

The wind resistant design principles for cable stayed bridges consist of the following:

1) With design wind loading and other actions, the bridge shall have sufficient strength, rigidity and shall meet the requirements of static stability.

2) During the service life of a bridge, it shall not be prone to divergent amplitude oscillations such as flutter and galloping

3) A low amplitude of oscillations due to vortices/ buffeting shall meet the requirements of traffic safety, structural fatigue and comfort of road users.

4) Wind resistant design shall be achieved by considering geometrical, structural or mechanical countermeasures.

5) The wind tunnel test is one important method to carry out the wind-resistant design of bridge structures (**Section 9.10.2**).

9.6 Conceptual Design Practices

The following aspects may be considered while selecting the deck, tower and cable type and geometry so that the wind effects are minimized. **[4]**

i) For long-span cable-stayed bridges, deck, tower and cable shall be chosen with consideration for wind-resistant stability.

ii) Cable-stayed bridges provided with single plane cable system along the centre line have usually a low torsional resistance, and are likely to be prone to aerodynamic instability named torsional divergence. For highway bridges with single plane cable system, single box girder is a standard solution for deck on account of its high torsional rigidity.

iii) For highway bridges with double plane system, depending on the span and width of bridge the selection of deck can be made (**Chapter 4**).

iv) For spans up to 200 m with deck width in the range of 15 m to 20 m, simple solid concrete slab without edge beams can be a good solution, while for wider decks between 20 m to 25 m, and spans in the range of 200 m to 500 m, T or I-beam supported deck with cross beams is expected to be more suitable.

v) Concrete cable stayed bridges with cables in two-planes along the edges are considered to be stable with respect to wind action if B≥ 10d (where B= breadth of deck and d= deck depth). Alternatively, if B <10d, then bridge deck with wind nose or fairing is preferred. Concrete decks with B≥ L/30 are considered favourable under wind action.**[5]**

vi) Steel bridges with main spans more than 400 m, provided with lightweight deck are more susceptible to wind-induced oscillations and A-shaped tower is

preferred. Steel decks with width B≥ L/25 is favourable with respect to wind action, alternatively provide wind nose or fairing if B< L/25.

vii) In the case of a truss-stiffened deck, plate girder supported decks, box –girder decks, good aerodynamic performance of bridge deck could be achieved by the use of shallow sections, closed sections, edge streamlining, and other minor and more subtle changes to the cross-sectional geometry.

viii) To increase the structural safety of bridge deck due to vortex induced fatigue, it is advantageous to make the onset of wind velocity for vortex excitation as high as possible. To achieve this condition, the bridge deck section should be as flat as possible but it should have sufficient stiffness to increase the natural frequency.

ix) Buffeting response of bridge deck to fluctuating wind depends on the steady-state drag (C_D), lift (C_L) and moment (C_M) coefficients as well as the derivative of lift ($dC_L/d\alpha_w$) and moment coefficient ($dC_M/d\alpha_w$) corresponding to angle of attack of wind. By selecting a cross-section for the bridge deck such that these aerodynamic design parameters and their derivatives are minimized, it could lead to a lower buffeting response.

x) Flutter stability of cable stayed bridge deck could be greatly increased by separation of vertical and torsional modes (f_t/f_v >2, where f_t and f_v are the first torsional and vertical mode frequencies of bridge deck respectively), high torsional rigidity for deck, and by adopting stream-lined bridge deck cross-sections.

xi) Provision of high torsional stiffness by the use of box sections for bridge deck and
A-shaped towers can help in raising the critical wind speeds for flutter to safe levels. The A- shaped tower gives a triangular shape of the cable plane and the deck, which increases torsional rigidity.

xii) For long-span bridges located in zones of high winds, A- shaped tower is the ideal solution to achieve wind stability. The upper level and deck level struts connecting the tower legs further help to improve the resistance in torsion.

xiii) Cables used in conjunction with cable pipes with surface treatment are preferred.

9.7 Desktop Study for Checking the Aerodynamic Susceptibility (Stage1)

The wind effects on cable stayed 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 CSB 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 (iii) Deck depth (d) at mid span and at the location of pylon.

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 (iii) Terrain category at the location of bridge as per IS:875: Part 3:2015 **[2]** (iv) Turbulent intensity (I_u) at the level of bridge deck, which depends on the terrain category can be computed. Also, compute the design wind speed at the location of bridge, taking into account the effects of terrain, topography and cyclonic conditions (V_d based on 3sec gust) at the bridge deck level as per IS:875 (Part 3) :2015 **[2].**

Step 2: Checking the susceptibility for dynamic wind action as given in Table 9.2 for bridge deck

Table 9.2 Checking the Susceptibility of Bridge Deck to dynamic wind effects (based on ref. [6])

Type of Bridge deck cross-section	Type of Wind induced oscillations of deck	Susceptibility Parameter	Limiting value	Inference	Further Action Required
Closed solid deck cross-section	Single degree of flutter in Torsion	L Vd/B	L Vd/B > 780	Deck is susceptible to torsional flutter	See note (i) to (iii) below. Also, perform check as stipulated in 10.8 if main span L > 300m
Truss, open solid deck cross-section			L Vd/B > 530		
Truss, open solid, closed solid deck cross-sections	Galloping	LVd/B and B/d	For composite decks LVd/B > 475 and B/d< 5	Deck is susceptible to galloping	See note (i) and (iv). Also, perform check as stipulated in Section 10.8
Truss ,open solid, closed solid deck cross-sections	Vortex induced oscillation	LVd/B and Iu	LVd/B > 275 and Iu < 0.2	Deck is susceptible to vortex induced oscillation	See Note (i) and ((vi)and perform check as stipulated in **Section 9.8**
For checking the susceptibility of bridge deck against buffeting, see note (i) and (v) below. When the fundamental frequencies in both bending and torsion estimated for the bridge components are greater than 1.0 Hz, the effects of turbulence response shall be ignored.					

Notes

i) If the bridge deck is susceptible to wind action as shown in **Table 9.1**, a free vibration analysis of the bridge subjected to dead load and SIDL is carried out using 3D-modelling 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. Otherwise, the wind speed for initiation of classical flutter shall be estimated and compared with design wind speed at bridge site.

iii) The critical wind speed for classical flutter V_f shall be computed from the reduced critical wind speed (V_{Rf}) **[7]**

$$V_{Rf} = V_f/(f_t\ B) \quad (9.1)$$

$$\text{where } V_{Rf} = 1.8\ (1-1.11(f_v/f_t)^2)^{1/2}\ (mr/\rho B^3)^{1/2} \quad (9.2)$$

r- polar radius of gyration of effective bridge deck cross-section at the centre of main-span (polar second moment of mass/mass)$^{0.5}$

The bridge deck shall be stable with respect to divergent response in wind storms if Vf >1.65 design wind speed (V_d) (based on 3 sec gust) at bridge location.

Compute the critical wind speed for initiation of single degree of freedom (sdof) 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 300 m may be calculated according to the following equation:

$V_{crf} = 5f_t \cdot B$, 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 (3 sec), 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 angles of attack. 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) The wind in the vicinity of earth's surface is turbulent. Turbulence causes buffeting and leads to fluctuating forces and moments on the bridge components. This action of the turbulent wind is experienced over a wide range of frequencies. For bridge decks with fundamental frequencies in respective bending (vertical), torsion or lateral directions <1.0 Hz buffeting response to be considered. Alternatively for bridge decks with span-to-depth or span-to-width ratios greater than 30, buffeting effect must be taken into consideration for aerodynamic design.

vi) Identify the vertical modes of bridge deck with frequency less than 1.0 Hz by inspecting the various mode shapes of vibration, and, 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, for truss /open solid deck the Strouhal Number is in the range 0.1 to 0.2.

If the computed V_{cr} values are lower than design wind speed at the deck level, then these modes are prone to vortex excitation in the vertical direction. 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.

In addition, follow the guidelines given below:

a) The effect of vortex resonance may not be considered when the fundamental frequency of structural component is larger than 5 Hz.

b) Twin box-girder bridges with an open gap are particularly sensitive to vortex induced motions. Vortices shed from the windward box combine with the vortices shed from the leeward box to increase the wind load on the leeward box, making a twin box configuration more sensitive to vortex induced motions than a single box configuration.

The Strouhal Number S = f d/V_{cr} defines the wind speed when vortex excited motions may occur. See **Table 9.3** for Strouhal number of a twin box-girder.

The gap ratio is defined as the gap width divided by the overall width of the twin boxes.

Table 9.3 : Gap Ratio Vs Strouhal Number

Case	Gap Ratio	Strouhal Number
1	0.000	(no vortex induced motion)
2	0.025	0.13
3	0.160	0.23
4	0.27	0.27
5	0.35	0.28

c) The vortex induced oscillatory amplitude in vertical direction of a box girder bridge deck may be computed according to the following equation [8]:

$$h_a = \frac{E_h E_{th}\, B}{2\pi m_r \zeta_s} \qquad (9.3)$$

where, $m_r = \frac{m}{\rho B^2}$, $E_h = 0.065\, \beta_{ds}\, \frac{d}{B}$, $E_{th} = 1 - 1.5\, \beta_t \sqrt{\frac{B}{d}}\, I_u^2 \geq 0$ and $I_u = \frac{1}{ln\frac{Z}{z_0}}$

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 stay 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 **[2]** till it is included in IRC:6 **[1]**

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. If yes, then, consider geometric modification of bridge deck or by installing a TMD

(d) Check the static stability

The wind velocity, called the divergence velocity V_{dg}, corresponds to the occurrence of a static stability problem. The divergence wind speed can be calculated by using the aerodynamic moment coefficient C_M of the deck for varying wind angles of attack α, by using Eq. 10.4 **[9]**.

$$V_{dg} = \sqrt{\frac{2\,K_\alpha}{\rho B^2 \frac{dC_M}{d\alpha}}} \qquad (9.4)$$

For bridge deck to be stable under wind action, $V_{dg} > 2\,V_d$

Step 3: Susceptibility of Tower to Dynamic Wind Action

Towers of cable stayed bridges 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) below Table 9.1 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

$V_{cr}=f_p D_p/ S$ (9.5)

If the height of free-standing pylon (i.e., during construction stage) is more than 30 m, then safety against dynamic wind shall be considered.

Step 4: Check for Wind induced cable vibration

The wind-induced vibration of stay 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 /tower anchorage, but also cause discomfort to bridge users.

Therefore, it is necessary to check the susceptibility of vibration of stay cable and incorporate countermeasures to mitigate the dynamic wind effects.

Dynamic effects of the wind especially in cables with length greater than 65 m 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:

$V_{cr}= f_c d_c/S$ (9.6)

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. 9.7. The Scruton number for stay cables without any dampers is typically Sc less than 1. However, if S_c is more than 2.5, then vortex induced oscillation is restricted.

VIV can be mitigated by reducing or eliminating the underlying excitation along the lengths of the 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 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 wind velocity range is around 6 to 18 m/s. The wind must be from an oblique direction, between 30° and 80° 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 **[7]**

$$S_C = \frac{m\xi}{\rho {d_c}^2} \geq 10 \qquad (9.7)$$

where

m - cable mass per unit length (kg/m) $\xi = \frac{\delta}{2\pi}$ and δ is logarithmic decrement of cable. For preliminary estimation, the critical damping ratio may be taken as 0.1% 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.9.7.

Where 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 to 5 times of cable diameter. However, it is found that the condition does not occur for cable spacing larger than $5d_c$ where d_c is the diameter of the cable.

9.8 Desktop study for Predicting Aerodynamic Susceptibility (Stage 2)

Wind resistant design for short to medium span bridges may be carried out by considering quasi-static wind effects, whereas longer span bridges necessarily exhibit a marked

aerodynamic behavior. Most cable-stayed bridges, being inherently flexible, are in the latter category. Since geometrical proportions of the bridge and the wind climate at its location too will influence the behavior of the bridge, a strict dividing line based on span alone may not be appropriate. Perhaps a dividing span length between 150 and 250 m may be more prudent from the point of view of these guidelines. Even for wind-sensitive longer spans, the quasi-static approach may be used for preliminary design and proportioning, before aerodynamic considerations are addressed.

Long span bridges; especially cable suspended bridges being flexible are prone to wind induced oscillations, especially due to gusts. These bridges undergo several forms of aerodynamic oscillations, which may result in motions in isolated vertical bending or torsional modes, or sometimes in coupled vertical bending and torsional modes.

Depending on the nature of excitation, the motion may of the following types for the bridge deck:

i) Limited amplitudes due to vortex-induced oscillations or turbulence in approaching wind, termed as buffeting, cause unacceptable stresses or fatigue damage. These type of oscillations shall be considered, during the design.

ii) Divergent amplitudes increasing exponentially to larger values, due to flutter or galloping, which must be avoided.

iii) Non-oscillatory divergence due to a form of aerodynamic torsional instability which must be avoided.

Aerodynamic susceptibility of the bridge may be checked as follows:

Method 1: Check whether the bridge is aerodynamically sensitive by computing the aerodynamic susceptibility parameter (P_b) **[7]**. It is stipulated that,

i) If the parameter P_b, given in Eqn. 9.8, is between 0.04 and 1, the bridge is prone to aerodynamic oscillations.

$$P_b = \left(\rho \frac{B^2}{m}\right)\left(16 \frac{(V_H)^2}{BLf_v^2}\right) \quad (9.8)$$

where

ρ =mass density of air (1.2 Kg/m^3)

B= deck width edge to edge

L = main span length of the bridge

V_H = hourly mean wind velocity corresponding to 50-year return period

f_v =natural frequency of the bridge in vertical bending mode in Hz

m=mass per unit length of deck

Case1: If P_b <0.04 the bridge will have insignificant effect of all forms of aerodynamic excitation

Case 2: If 0.04 ≤P_b <1.0 then the bridge deck is aerodynamically susceptible

Case 3: If P_b >1.0 the bridge deck is potentially very susceptible for aerodynamic excitation (bridges with main span more than 250 m are likely to fall in this category)

Method 2: Bridges and bridge components are deemed wind-sensitive **[10]**, when the mass damping parameter Sw < 1.0,

where Sw = (Sc) (B/L) (B/d) (f_v B/ $k_1 x V_b$) (9.9)

and Sc = $m\zeta/\rho d^2$

m - mass per deck length

ζ - structural damping ratio

ρ - air density

d - deck depth

B - deck width edge to edge

L - main span length

f_v - lowest frequency in vertical mode

V_b – basic wind speed at bridge location

It is imperative that even if one of the above methods indicates a need for aerodynamic investigations, these should be pursued. Furthermore, whatever may be the indication, for bridges with more than 250 m main span, aerodynamic investigations including wind tunnel testing should be carried out.

If a bridge site is windy, such as locations on the coasts of India, or the hilly regions where deep valleys can experience accentuation of wind speed, the project should, in any case, consider including aerodynamic investigations. It will be prudent to carry out a wind climate assessment which will provide more detailed information on the wind climate around the bridge site. This will be expected to follow a ‘desktop analysis’ for the bridge, and, if so indicated a wind tunnel study be undertaken.

9.9 Data Required for Aerodynamic Design

9.9.1 *Geometric Details*

Geometric details of bridge deck, cable and tower are required for modelling purpose. Also, it should include the geometric details of railing, crash barrier, noise barrier and other components which are essential for numerical modeling /wind tunnel studies.

9.9.2 *Material Properties*

The material used for construction of deck, tower and cable, their density ,modulus of elasticity, mass per unit length, poisson's ratio etc. are needed.

9.9.3 *Damping Ratio for Bridge Deck and Tower*

The following structural damping ratios (percentage of critical damping) depending on the material to be used, are recommended for analysis of wind effects, including aerodynamic stability analyses:

- Steel bridges – 0.5%
- Composite steel/concrete bridges – 0.7%
- Concrete bridges – 0.9%

For bridges under construction, reduce the above recommended values by 0.2%

9.9.3.1 Damping Ratio for Cables

For preliminary estimation, the critical damping ratio may be taken as 0.1% for all type of cables. For better estimates see **Table 9.4** or the values certified by the cable manufacturer.

Table 9.4 Typical damping of Stay Cable System

Cable Stay Type	Critical damping ratio
Grouted stay	0.08 to 0.13%
Locked coil	0.15 to 0.3%
Stay cable provided with HDPE duct, galvanized strands and filled with wax	0.1 to 0.18%
Stay cable provided with HDPE Duct and individually protected strands (Freyssinet)	0.1to 0.20*

*Damping factor = $-6x10^{-4}$ L + 0.24 (where L is length of stay in m)

9.9.4 *Wind Characteristics at Bridge Site*

To determine wind effects on cable-stayed bridges, it is stipulated that the basic relevant information about wind will be obtained. Bridges in the cyclonic regions will be treated as life-line structures of post-disaster importance. Since wind clause of IRC:6 **[1]** is based on IS:875: Part 3 **[2]**, the same shall be referred for description of wind.

The wind speeds as found in contemporary literature or documents are commonly based on different averaging times and return periods. In this context, certain conversion factors (k_1 Factors) have been included for the design. For the structural design of cable stayed bridge

a return period of 100 years and for the aerodynamic stability check the return period of 10, 000 years shall be used. For construction stage analysis, a return period of 10 years shall be used **Table 9.5**.

Table 9.5. Multiplier Factors, k_1, for Velocity V_b for Different Return Periods (V_b m/s for Return Period of 50 years as given in [2])

Return period of wind (years)	k_1 factor for basic wind speed (m/s)					
	33	39	44	47	50	55
5	0.82	0.76	0.73	0.71	0.70	0.67
10	0.87	0.83	0.81	0.8	0.79	0.77
25	0.94	0.92	0.91	0.91	0.9	0.89
50	1.0	1.0	1.0	1.0	1.0	1.0
100	1.05	1.06	1.07	1.07	1.08	1.08
500	1.17	1.22	1.25	1.27	1.28	1.30
1000	1.23	1.29	1.33	1.35	1.37	1.40
10000	1.40	1.52	1.59	1.63	1.66	1.71

9.9.5 *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 joints (with and without live load at service and construction stages) (ii) Tower (iii) Stay cable system

9.9.5.1 Bridge Deck

Static wind forces on the deck F_D, F_L and F_M can be computed in terms of the force coefficients C_D, C_L and C_M **Fig. 9.6** representing drag, lift and pitching moment respectively for the deck. These coefficients are to a great extent dependent upon the shape of the deck as well as the angle of incidence of wind (measured in the vertical plane). Some typical deck cross-sections and the corresponding values of C_D are shown in **Fig. 9.7.**

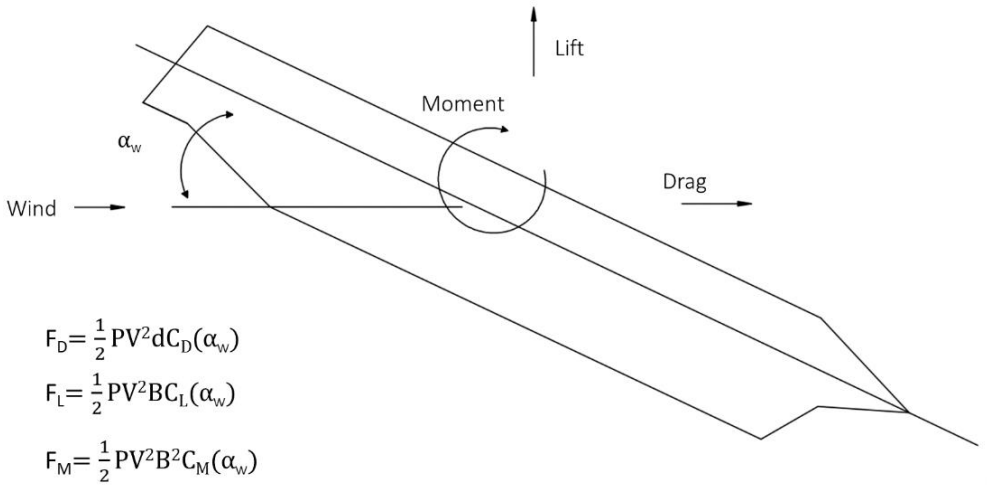


Fig. 9.6

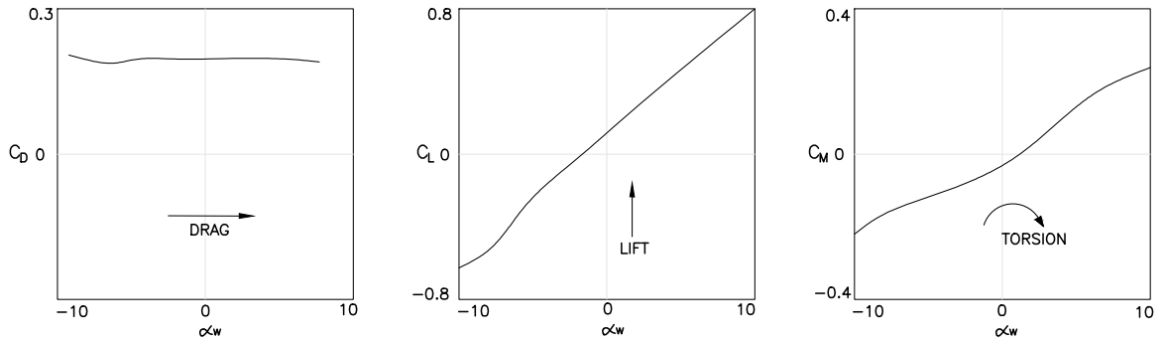


Fig. 9.7 Typical drag, lift and moment coefficients for a bridge deck

For bridges which are not sensitive to dynamic wind effects, the force coefficients (drag and lift) as given in IRC:6 may be selected depending on the cross-sectional shape and aspect ratio (b/D) of bridge deck. However, IRC:6 **[1]** stipulates only drag and lift coefficients of bridge deck cross-section. Aerodynamic force coefficients (aerodynamic drag, lift and moment coefficients at wind angles $+6^0$ to -6^0) of bridge deck shall be obtained through numerical studies or wind tunnel studies.

The utility of mean wind force coefficients or aerodynamic coefficients is not limited in the determination of mean wind load. These coefficients and their derivatives with respect to the angle of attack ($dC_L/d\alpha$) and ($dC_M/d\alpha$) are used to determine buffeting forces and to check the galloping stability.

9.9.5.2 Tower

The wind effects are governed by the structural properties and geometric shape of the tower as well as the characteristics of approaching wind. For hourly wind speed and pressure variation along the height of the tower, turbulence intensity depending in the terrain category at the location of bridge IS:875: Part 3:2015 **[2]** shall be referred. The wind load acting on tower shall be determined for a wind speed at a height 0.65 times height of tower from mean ground/water surface.

The tower 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 obtained from free vibration analysis is more than 1.0 Hz.

For the computation of along wind forces, the drag coefficient for a 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 **[1]**.

If the conditions stipulated in a) and b) are not satisfied, the tower shall be prone to buffeting, and Gust factor method as given in IS:875: Part 3:2015 **[2]** 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.

Wind load on tower shall be computed at various stages of construction as well as for completed configuration.

Wind load on towers shall take into account dynamic and static wind pressure, buffeting, and vortex shedding.

9.9.5.3 Stay Cable System

Cables in a cable-stayed bridge are subject to substantial forces due to wind and this is transferred to their ends into the deck and the towers. For longer spans, the reaction on the tower may match the direct force it experiences due to wind. Stays in a cable stayed bridge are like strings under tension.

For circular sections, such as for a cable, the force coefficient depends on how the wind flows around it and is dependent upon the wind velocity, kinematic viscosity, diameter of the cable, and surface roughness. In general, the force coefficient for a circular section varies with Reynold's number (R_e) and surface roughness. The dependence of the force coefficient of a circular section on R_e is due to the change in the wake developed behind the body.

The drag force from the wind per unit length of the cable shall be calculated using Eq. 9.10.

$$F_D = \frac{1}{2} \rho V_Z d_c C_D G \qquad (9.10)$$

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 $d_c V_z <$ 6m^2/s and 0.8 if $d_c V_z >$ 6m^2/s.

9.9.6 *Buffeting Analysis*

Buffeting analysis of the cable stayed bridge in frequency domain or time domain shall be carried out, (if the steady state aerodynamic coefficients and their derivatives are obtained through wind tunnel tests or CFD) to calculate the peak amplitudes and modes of vibration at the design wind speed. By performing a three-dimensional buffeting analysis in time domain, considering the geometric non-linearity of cable elements (by considering the modified E value of cables) the relevant buffeting induced forces (Axial force, BM, SF) on bridge deck, tower and cables can be obtained. Wake buffeting or proximity effects shall also be evaluated for twin deck configurations or adjacent bridges.

9.9.7 *Strouhal Number*

Strouhal Number is required to compute the respective critical wind speed for vortex excitation of bridge deck, tower and cable.

In general, the Strouhal number of bridge deck cross-section varies between 0.1 and 0.2. Strouhal number for box girder supported deck of cable stayed bridges varies in the range of 0.1 to 0.15. This shall be verified using numerical modeling or wind tunnel studies when span of bridge is > 250 m and or when the bridge deck is sensitive to vortex induced oscillation as stipulated in Section 9.7 and 9.8.

For circular cables the Strouhal Number shall be taken as 0.2. For square or rectangular cross-section of tower Strouhal number shall be taken as 0.12.

9.9.8 *Flutter Derivatives*

Flutter is a self-feeding aeroelastic phenomenon which is potentially destructive to long-span cable-supported bridges. The aerodynamic forces generated on the bridge deck couple with its motion and, if the energy input by the aerodynamic forces at high wind speeds in an oscillation cycle becomes larger than that dissipated by the mechanical damping of the bridge structural system, the amplitude of vibration will grow. This increasing vibration will then amplify the aerodynamic forces, resulting in continuously growing self-excited forces and self-exciting oscillations. If this situation continues for some time, it can lead to the ultimate limit state of the bridge and ultimately to collapse.

Flutter derivatives represent the characteristics of self-excited aerodynamic forces of a bridge deck cross-section and are the most important parameters used to identify the incipient stage of flutter, with the estimated critical wind speed for onset of flutter using these derivatives. Flutter derivatives are generally determined by means of wind tunnel studies.

Reference **[11]** can be perused for providing detailed information and understanding about the application of flutter derivatives to the study of aerodynamic stability of long span bridges.

9.10 Investigations for Design Parameters

The aerodynamic parameters required for cable stayed bridges shall be obtained through (i) numerical simulation studies and or (ii) wind tunnel studies

9.10.1 *Numerical Simulation/CFD Studies*

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.

9.10.2 *Wind Tunnel Studies*

Wind tunnel studies on sectional model shall be carried out for Cable stayed bridge deck if the criteria given in **Table 9.2** is not satisfied and or aerodynamically found sensitive as stipulated in **Section 9.8**, and, also for Cable stayed bridge with main span more than 250 m 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.

(vi) Aerodynamic derivatives used to determine flutter speed analytically shall be extracted from testing at wind speeds beyond the flutter speed criteria. At least 8 derivatives shall be identified for 2 degree-of-freedom (2-dof) tests, and 18 derivatives shall be identified from 3-dof tests.

(vii) If a free vibration method is used, aerodynamic derivatives shall be extracted in smooth flow.

(viii) A full model should therefore be used only where non-uniformity of section or of incident flow conditions, complex dynamics or erection considerations necessitate its use.

(ix) The following typical scales for the various types of wind tunnel tests are recommended:

Topographic models 1:2000 (To examine the influence of topography or other perturbations)

Aero-elastic models 1:200 to 1:100

Section models (stability or time average coefficients) 1:80 to 1:40

However, for the range of span of cable stayed bridges in focus in these guidelines (upto 500 m), it should commonly not be required to deploy a full aeroelastic model.

Some additional information is given in **Annexure A3**. Though for carrying out wind tunnel model testing of a bridge, essentially, specialist literature, and, preferably also an experienced wind tunnel establishment should be consulted.

9.11 Mitigation Measures

Mitigation measures for aerodynamic design of cable stayed bridge include either or combination of the following modification such as (i) geometric (ii) structural and (iii) mechanical means.**[4]**

One or more of the measures mentioned above to augment the aerodynamic stability of the bridge deck system, have mostly to be based on experience and good practice rather than any theoretical analysis. Essentially, these have to be proved through model testing. Furthermore, these measures do not necessarily have to be provided throughout the length of the span, and, partial treatment of parts of the span may suffice.

9.11.1 *Deck*

i) The vortex-induced response of a bridge deck can be influenced by a tiny change in the configuration of the leading and trailing edge of the cross-section. Similarly, the installation of the solid handrail / crash barrier on the deck section may increase aerodynamic instability. The horizontal member of the handrail can play an important role in the vortex-induced oscillation.

ii) New Jersey type crash barrier used in cable-stayed bridge decks acts as a “flow tripper” and causes vortex-induced motion. Therefore, it is imperative for designers contemplating the use of this type of crash barrier to perform wind–tunnel tests for confirming the stability of the proposed deck cross-sectional configuration. In fact, 50 to 70% porous steel crash barriers can improve the stability performance of the deck significantly and has been implemented on many bridges around the world.

iii) The two efficient ways to suppress the vortex-induced oscillations are to restrict the formation of a separation bubble very close to the leading edge of the bridge deck or make it spread very far of the wake. Practically, this is achieved with the help of aerodynamic appendages such as the provision of Baffle Plates, Fairings on part of the span, for example shown in **Fig. 9.8** (a) & (b), to disorganize the wind and disturb the rhythm with which vortices may be shed.

iv) A box girder deck section, symmetrical about the vertical center plane and featuring rounded off edges or use of streamlined box section, with a wind nose is preferable to minimize the effect of vortex-induced oscillation. In case, unfavourable downstream flow develops, it can be compensated by the introduction of guide vanes **Fig. 9.8 (c).** The aerodynamic functions of guide vanes are as follows:

   (a) To reduce or avoid the formation of coherent large-scale vortices in the wake of girder due to relatively abrupt change of surface angle, and thus eliminate vortex-induced vibrations at low speeds.

(b) To increase the aerodynamic damping in torsion and thus enhance the critical wind speed for vortex shedding.

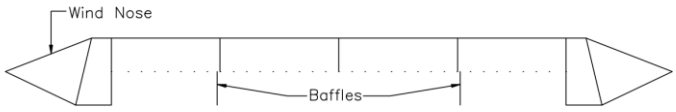


Fig.9.8 (a) Baffle plates

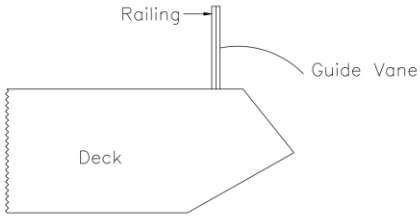


Fig. 9.8 (c) Guide vanes

v) Accumulation of ice or snow may effectively change the aerodynamic shape in the critical areas of bridge deck-like edges and can alter the response of the bridge deck due to vortex-induced oscillation. The presence of movement of traffic on the bridge can also alter the response due to vortex shedding.

vi) In an open–slotted bridge deck by providing a lateral bracing system torsional strength could be enhanced thereby increasing the aerodynamic stability.

vii) The aerodynamic instability in long-span bridges may be reduced, and in some cases eliminated, through the use of proper combination of slots (openings on the bridge deck floor) and fins (lateral projections).This is because the lateral or central slots eliminate vertical instability and outside fins reduce or eliminate torsional instability. A combination of the above two in suitable proportions has been found to have yielded good results for both vertical and torsional instability.

viii) Critical velocity for the flutter of bridge deck with H, $\prod$ and flat plate type cross-section can be enhanced by the installation of open grating in the deck. Further, it is better to locate the open grating as close to the edge as possible in the section with an area of opening of more than 30% of the deck area.

ix) Bridge decks with impermeable railing are likely to be prone to a single degree of freedom flutter in torsion. Since railings are required to guard the traffic, they should be made permeable for better aerodynamic stability.

x) Aerodynamic stability of super long-span bridges could be achieved by controlling the stiffness distribution along the span of bridge deck i.e. by flutter mode shape control.

xi) he critical flutter wind speed of long- span flexible bridges could be increased by the installation of tuned mass dampers.

xii) Active control systems are found useful in wind sensitive bridges to enhance the comfort of the bridge users and to reduce fatigue damage.

xiii) The connection between deck and tower, i.e., either a floating deck of elastically supported connection between the deck and tower are appropriate for aerodynamic consideration.

xiv) A damper could be used as a temporary measure to prevent vortex-induced oscillation of a cable-stayed bridge under construction.

9.11.2 *Tower*

The improved aerodynamic design of tower is achieved with the help of corner deflectors, central slit in cross-section as well as with corner modifications **Fig.9.9**.

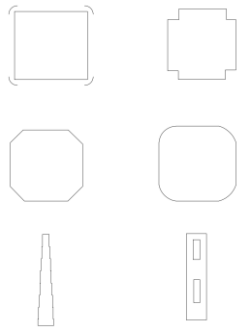


Fig. 9.9 Geometric Modifications /Corner Modifications of Tower

Geometric modification of tower cross-section such as introducing corner-cut is one of the well-known methods to reduce the vortex-induced vibration of tower with rectangular sections **[12]**

Wind-induced vibration during erection affects the safety and serviceability of the structure; typical examples are pylons in the free-standing stage. In tall pylons, not only the first fundamental mode but also higher modes need to be checked and controlled, if needed.

Tall steel pylons require temporal control for wind-induced vibrations In such situations, mechanical control such as sliding-block-type dampers were installed (by connecting to the top of pylons by wires) to control the vortex-induced vibration of pylons of bridge during the erection. Also, applications can be found in long-span cable-supported bridges including Yokohama Bay Bridge, in which viscous dampers were employed. Though TMD and TLD are useful in controlling vibrations, they are effective only in small band of frequency **[13]**. TMD was also used to control the cantilevered girder of a cable-stayed bridge during erection. By considering cost, maintenance and reliability aspects passive TMD’s are preferred instead of active control systems.

Furthermore, to alleviate the wind-induced oscillatory problems of the tower during operation stage, Tuned Mass Dampers (TMD) could be installed.

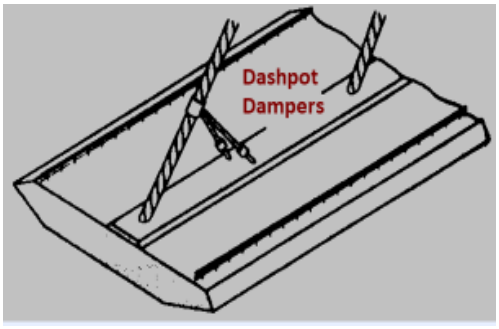
9.11.3 *Cables*

i) In case of cables which are found to be susceptible to rain-wind induced oscillation, surface modification of HDPE duct could be adopted. Surface modifications, **Fig. 9.10** such as double helical rib or incorporation of dimples on the HDPE duct keeps the stay cable stable by preventing the occurrence of the water rivulet under certain wind conditions. This is a preferable solution as it does not increase the drag of the cable while maintaining the required aesthetical appearance of the stays. However, its efficacy will have to be tested by wind tunnel testing, unless the information is found in specialist literature.

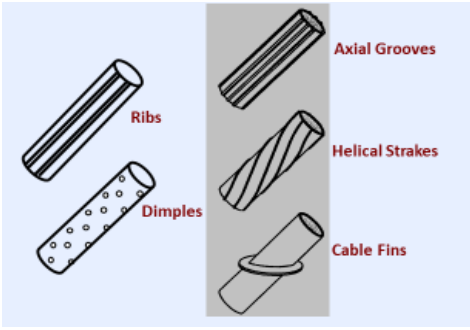
ii) Structural control of vibrations by interconnecting cable-ties has been used very frequently, either temporarily, before the installation of damping devices **[3]** on the cables, or as a permanent measure.

iii) Cross ties are recommended when the vibration frequencies of the deck or tower are close to the frequencies of the stay cables. They consist of a bundle of stainless-steel wire ropes fully embedded in HDPE and other dampening plastic material. They are to be tested for fatigue and damping capacity. A steel collar provides the fixation of the ropes on each stay and facilities for any orientation.

In terms of structural behaviour, the addition of cross-ties to the stay cable system creates intermediate supports at those elements and, consequently, increases their natural frequencies for vertical vibrations. With a proper design of these ropes, the fundamental frequency of the stay cables is shifted to relatively high values, thus reducing the possibility of internal resonance with global bridge modes at the fundamental mode shapes. Another effect of the installation of cross-cables is an increase of the damping. **(Table 9.6)**



(a) Dampers Installed on the Stay



(b) Surface Treatment to Control Rain-Wind Oscillations

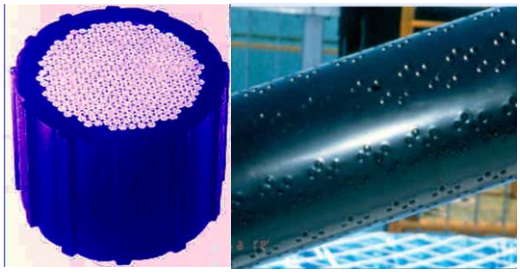


Fig. 9.10 Illustrating Measures for Controlling Stay Oscillations in Stay cable

Table 9.6 Stay Cable Vibration Mitigation Measures

Sl no.	Bridge Name	Year of construction	Main Span (m)	Cable vibration observed	Oscillatory amplitude (m)	Mitigation measure adopted
1	Kohlbrandt, Germany	1974	326		1.0	Viscous dampers
2	Tjorn, Sweden	1981	360	Vibration during light rain	-	Viscous dampers
3	Brotonne, France	1977	320	Vibrating in 15m/s wind	0.6	Viscous dampers
4	Ben-Ahin Belgium	1987	168	Vibration noted during light drizzle and 10 m/s winds	1.0	Petroleum wax grout added in the guide pipe and cross cables
5	Yobuko	1988	250	Vibration during light rain	0.15	Manila ropes attached to the cables
6	Aratsu Japan	1988	186	Vibration during light rain	0.6	Viscous dampers
7	Wandre. Belgium	1989	168	Vibration during light rain and 10m/s winds	0.5	Petroleum wax fill in the guide pipe
8	Alzette, Luxembourg	1993	130	Vibration during drizzle and light wind	-	Neoprene guides inside the guide pipe at deck and petroleum wax fill in the guide pipe
9	Weirton Steubenville USA	1990	250	Vibrations noted when winds are parallel to deck	0.6	Visco-elastic dampers installed in the guide pipe at deck level
10	Tempozan	1990	351	Vibration during rain and 10m/s winds	2.0	Viscous dampers
11	Helgeland Norway	1991	427	Large cable vibrations; depending on deck motion	0.6	Cross-cables
12	Normandie France	1995	853	Cable vibrations during steady 1 to 2m/s winds	1.5	Viscous dampers
13	Second Severn	1997	457	Cable vibration with and without rain	1.5	Cross cables
14	Meiko Nishi	1997	404	Vibration during light rain and low winds	0.55	Cross cables
15	Anzac Australia	1995	345	Vibration during light rain and wind	1.5	Dampers/ surface treatment of cables

9.11.4 *Use of dampers*

Use of dampers is unlikey to be there for cables up to 60 m length. For cables above 60 m length additional damping requirement can be worked out using Scruton number (Equation 9.7).

Both external or internal dampers could be adopted to enhance the damping ratio of the cables. External damper may be hydraulic damper and are visible and requires maintenance. Hence, not preferred in comparison to internal dampers which are not visible and maintenance free.

Internal dampers are of two types- (i) Internal Elastomeric type (IED) generally used for cables up to 150 m length and for cables with more than 150 m length (ii) Internal Hydraulic dampers (IHD) or a combination of IED with IHD may be used.

Internal Elastomeric Damper (IED)

A cast piece of special rubber with high damping characteristics and a specially designed shape is placed between the stay cable and the guide tube (deck and pylon). If the damping characteristics need to be increased, viscous oil can be placed in a special chamber. This is a very compact system with virtually no maintenance. It can provide sufficient damping for cables as long as 150 m. This damper is completely invisible from the deck. For details **Chapter 11** shall be referred.

Internal Hydraulic Damper (IHD)

Internal Hydraulic Damper (IHD) is generally used for cables longer than 150 m. It may be used in parallel with the IED, to increase the damping characteristics of the whole system. It has the same advantage as that of IED, i.e., it is not visible and almost maintenance free.

Limiting Value of Cable Vibration Amplitude-Comfort Criteria

Study on sensitivity of bridge users to cable vibration has indicated the comfort criteria for cable displacement can be described as follows for cables with frequency in the range 0.5 to 2 Hz. The cable displacement shall be kept below one d_c (d_c is the cable diameter) and no case shall exceed 2.0 dc **[10]**.

Wind Related Terms & Notation

1. **Bluff body:** A bluff body can be defined as a body that as a result of its cross-sectional shape has a separated flow over a substantial portion of its surface, resulting in the formation of a wake having significant lateral dimensions and unsteady velocity fields.

2. **Turbulence:** Turbulence in wind originates from the action of shear stress at ground level due to surface roughness and ground obstructions, which are transmitted upwards through the atmosphere leading to the formation and decay of eddies.

3. **Fairings:** Fairings are appendages of triangular or circular elements attached externally at the leading edges of a bluff bridge deck to enhance their aerodynamic performance, especially concerning vortex-induced oscillation, by altering the flow pattern around it.

4. **Guide Vanes:** Guide vanes or deflectors are provided at the corners of the bluff body (bridge deck or pylon) to modify the wind flow pattern by preventing flow separation at sharp corners, thereby mitigating vortex-induced vibration.

CHAPTER - 10

STAY CABLE SYSTEM, MATERIALS AND QUALITY ASSURANCE

10.1 General Requirements

10.1.1 *Principles for Cable Stay Protection*

The basic principle for a cable stay protection means that the cable stays 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 stay cables is always lesser than service life of structure. Considering the difficulties in replacement of stay cables, it is necessary that stay cables require to be replaced only once in the entire service life of structure. Appropriate maintenance is considered to be in order during the period before the stay has to be replaced.

10.1.1.1 Fretting –Resistance Precautions

The main tensile elements of which cable stays 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 to the stay or saddle zones. As a result of microscopic and frequent movements at these contacts, there is loss of material by wearing and removing oxides thus formed in powder form. The effects of these phenomena are called as 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.

10.1.1.2 Redundancy of Corrosion –Protection Barriers

Corrosion protection of the steel tensile elements of cable stays must be provided with a degree of redundancy by means of two complementary **Fig. 10.1**, 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.

10.1.1.3 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.

10.1.2 *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.

10.1.2.1 Protection by Metal Coating

The quality and lifetime of this internal protection depends 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 Z-shaped wires (locked-coil cables), comply with the dimensional tolerances that will ensure good wire interlock;
- 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.

10.1.2.2 Non-metallic Protection Equivalent to Galvanization

Instead of a galvanized metal coating, a protective coating using new materials (such as epoxy coating) 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 tension element under static (bending) and dynamic action effects in service;
- lubrication of contact points to prevent fretting corrosion and fatigue;
- corrosion protection.

10.1.3 *External Barrier*

The external barrier can take a variety of forms, in accordance with the type of cable stay 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 cable stay (outer casing and intermediary medium);
- The arrangements for ensuring the continuity of the barrier (intermediary medium) throughout the anchorage zone.

10.1.3.1 Watertight Outer Casing

For the free length of the cable stay, 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 cable stays
- 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 cable stays 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).

10.1.3.2 Intermediate Medium

In the free length of the cable stay, 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.

10.2 Stay Cable Types

In earlier days of construction of cable stayed bridges, parallel strand system with Outer HDPE pipe injected with cement grout (as an intermediate medium) were used. But, this proves to be a failure for following reasons:-

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

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

  - Parallel Strand System-PSS
  - New Parallel Wire System-NPWS
  - Locked Coil System (LCS)
  - Epoxy Coated And filled Strand (ECF strand)

  In the majority of recently constructed cable stayed bridges, Parallel Strand Cable (PSS) has been used.

10.2.1 *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 5 mm in diameter except the central wire which is of larger diameter than 5 mm. Tensile strengths of such strands available in market are, 1860 MPa for 15.2 mm and 1770 MPa / 1860 MPa for 15.7 mm dia **[6]**. However, 5 mm wires with higher tensile strength of 2000 MPa have been also developed.

Each wire of a strand is 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 being governed by numbers of strand grouped in it. Required diameter of the duct for PSS cable is more than other two stay cable types (PWS or MLS). Large diameter of outer HDPE results in more wind loading on structures which is very vital criteria for large span. In last decade, stay cable supplier's 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.

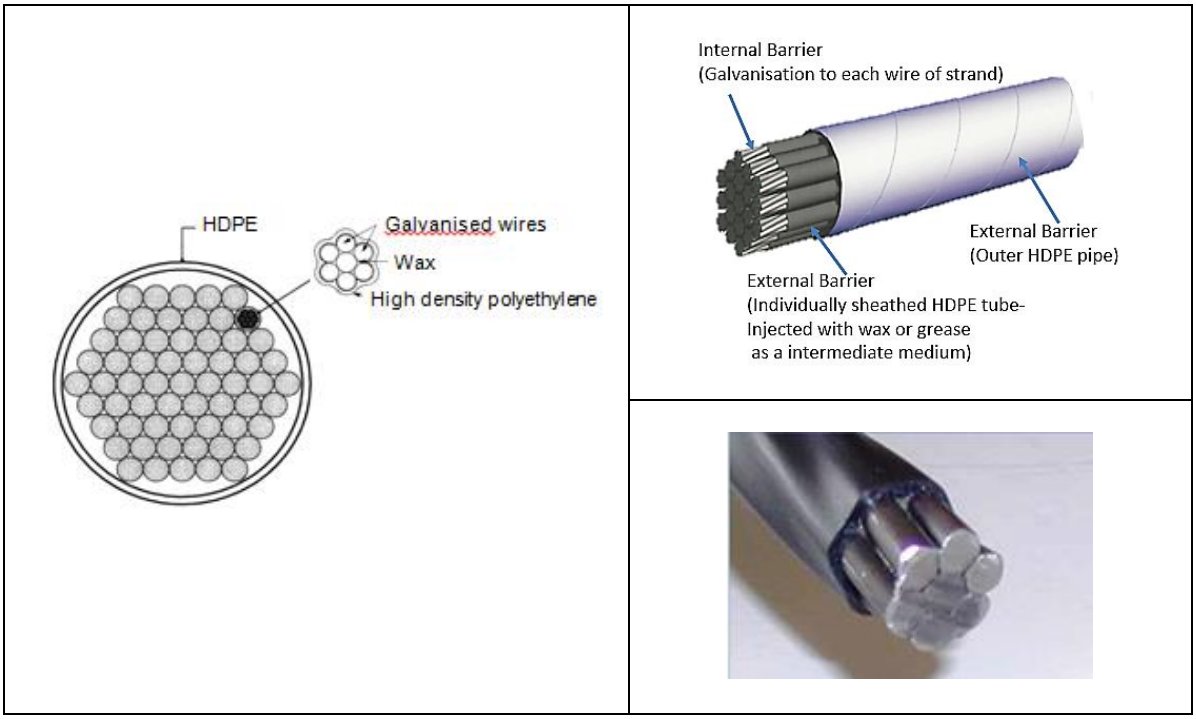


Fig. 10.1 Cross section of Parallel Strand System (PSS) cable

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

10.2.2 *New Parallel Wire System (NPWS)* ***(Fig. 10.2)***

NPWS cables are a development of PWS cables, which was developed and used in Japan in the 1970s and 80s. They 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 7 mm 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.

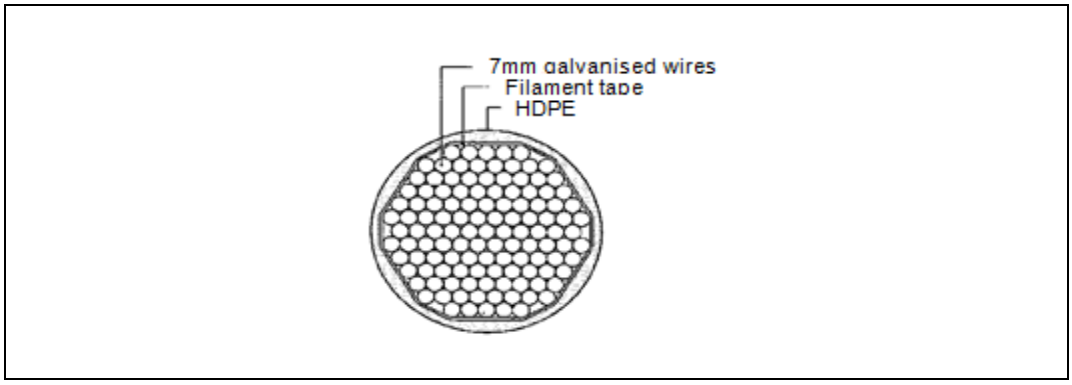


Fig. 10.2 Cross section of New Parallel Wire System (NPWS) cable

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

Diameter of whole cable ranges from 45 mm to 192 mm and the maximum breaking load for a 192 mm NPWS cable is 33.99 MN based on 1770 MPa tensile strength.

10.2.3 *Locked Coil System (LCS)* ***(Fig. 10.3)***

Locked coil cables are composed of two types of twisted wire.

In the core of cable, normal rounded wires are wound helically round a core wire. Such layers in inner core are wound in alternate directions.

In outer layer, wire are of a special Z-shape (2 to 4 layers of interlocked wires). The Z- shape is so chosen so that wires interlock and ensures a tight surface. Also the fit of the Z-wires leads to a very small void ratio inside the cables. The wires used for LCS cables generally have tensile strengths of 1370 MPa to 1570 MPa.

In all cases, all wires shall be galvanised. In most cases, this corrosion protection is only combined with the traditional surface treatment of painting. If required a polyethylene tube can be extruded onto the LCS cables in a similar manner as employed for NPWS cable. However, inspection and repair of painted cable is easier.

Such cables are manufactured in full length and full cross-section and delivered with sockets on large reels.

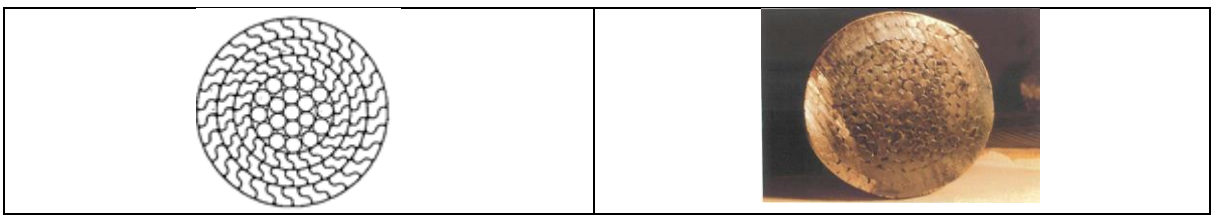


Fig. 10.3 Cross Section of a Locked Coil System (LCS) Cable

10.2.4 *Epoxy Coated and filled Strand (ECF strand)* ***(Fig. 10.4)***

Filled epoxy coated strands are available for seven wire assemblies **[7]**. The strands are coated with epoxy resin which is fully filled among each of the wires, giving its high corrosion protection and outstanding resistibility against fretting fatigue. Since, epoxy resin is not suitable for use in prolonged UV light radiation environments, PE sheathed ECF strand are recommended.

However, great care has to be exercised to prevent abrasion and damage to coating during its transportation and site installation. The most basic precaution needed is to prevent ECF strand from direct contact with something harder than the epoxy resin like metal or concrete.

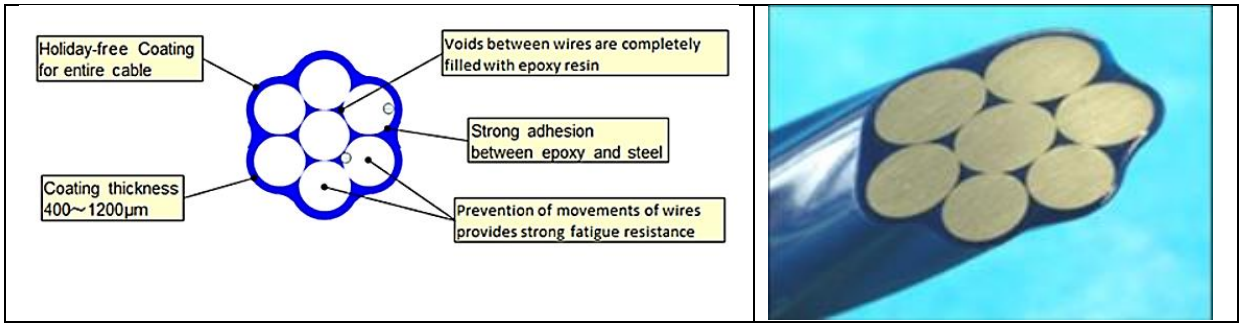


Fig. 10.4 Cross-section of Epoxy Coated and Filled Strands

Special wedges that bite through the epoxy coating and grip the prestressing steel are required. Wide & deep teeth are needed for good anchorages of the strand.

10.3 Qualification Criteria and Testing of Stay Cables System

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

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

The cable stay 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 long, the cable stays are more sensitive in cable stayed bridge than in a extradosed bridge, to fatigue effects and also to dynamic vibrations which also induce fatigue effects. Moreover, in the majority of cable stayed bridges, the stress variations are more significant than in a Extradosed Bridges.

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

If for a particular project all the cables have $\Delta\sigma_{sls}$ (stress range in cables due to vehicular live load under frequent live load in MPa) less than 50 MPa, the qualification testing (initial

and suitability testing) requirements of the extradosed bridges shall be adopted else all the requirements of the cable stayed bridges shall be adopted (**Annexure-2**).

10.3.1 *Laboratory and Independent Witnesses*

Initial approval testing or suitability testing shall be executed in a laboratory which fulfils the requirements provided in EN ISO/IEC17025 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.

10.3.2 *Initial Approval Testing (System Qualification testing)*

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

Stay cable system supplier tests (conducted in last 5 years) are considered as valid as long as there are no changes in relevant stay cable details.

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

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

- Three Anchorages to be tested for axial fatigue and tensile tests performed on typical sizes (one small, one medium, one large) **[2]**
- Three Anchorage bending 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 stay cable shall include :-

- One complete fully assembled stay cable specimen with saddle setup for saddle axial fatigue, friction and tensile test **[2]**
- One complete fully assembled stay cable specimen with saddle setup for saddle bending fatigue and subsequent 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.

10.3.3 *Suitability Testing*

Basic materials, components and detailing of the stay 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 cable stay system fabricated by a specific supplier and /or a process and intended for use on a particular project.

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

Suitability testing of a stay 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 stay-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 :-

- Stay-Saddle fatigue and tensile tests using saddle similar to as carried out for stay-anchorage axial fatigue and tensile test **[2]**.

- One friction tests 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 out in a sequence as mentioned above. Failure of any of the above test, may require re-testing of all previous tests.

Stay cable anchorage components have to be installed, while casting the segments. It is, therefore, preferable to have the testing completed prior to casting stay 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]**.

10.3.4 *Quality Control Testing*

The objective of quality control testing is to determine that the properties of materials and components installed in the cable stayed 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 A4** for testing of various material checks of strands,

sheathed duct, filler material and Outer HDPE duct, wedges and anchorages parts are required to be performed **[2]**.

10.4 Cable Vibrations and Control

10.4.1 *General*

The issues related to Cable Vibrations and Control have been addressed in **Chapter 9** dealing generally with Wind Effects and Aerodynamic Design Considerations. This is dealt with here briefly, more particularly to bring out control measures to be applied to stay cables.

As mentioned in **Chapter 9** also, vibrations may occur in stays due to vortex shedding, wake effects, buffeting and cable galloping. It is generally necessary to introduce additional damping in the cables to a level of 3 to 4% logarithmic decrement to suppress such vibrations. Two different types of damping systems are normally considered: internal dampers, installed in connection with the cable anchorage and external dampers, installed independently of the anchorage and fixed to the deck.

The use of internal dampers requires a stiff support at the damper position. For maximum efficiency the internal damper should be placed at a distance of at least 1.5% of the cable length from the anchorage. For a 500 m long cable this means a distance of 7.5 m between the damper and the anchorage point. This is normally best achieved if the cable is taken through the deck and anchored inside or below the deck with the damper placed above the deck and inside the stiff anchorage tube. Suppliers of PSS and NPWS cables have developed internal dampers of various types to suit their cables and anchorage layout. Measurements on several bridges have proved the effectiveness of these dampers. For LCS cables specific internal damper systems have, to the best our knowledge, not been developed.

The design of the cable system should allow for future installations of external dampers i.e. the designer should make sure that in case additional damping is required to suppress unexpected vibrations then a solution is at hand to mitigate them.

10.4.2 *Parametric Excitation*

Vibrations of the cables may also be introduced through structural vibrations at their anchorages either at deck level or tower level. These vibrations can be produced by different actions such as vortex shedding on deck or tower, buffeting or movement of traffic. If these forced vibrations of the anchorages reach a frequency, which coincide with a cable mode, then very large cable amplitudes can occur in the cable. This phenomenon is called parametric excitation. While damping can affect the peak amplitude reached by the cable, it is important to note that adding damping cannot prevent the excitation to take place. It may thus be more prudent to suppress the source.