10.4.3 *Rain-Wind Induced Vibrations*

It is common for stay cables to be subjected to rain-wind induced vibrations, as brought out in Sec.10.5.1 (**Chapter 9**). Both PSS and NPWS cables are prone to rain-wind induced vibrations. The diameter of sheaths is irrelevant but their surface condition plays an important role. Such vibrations are observed on painted steel sheaths, stainless steel sheaths and HDPE sheaths. If the surface of the sheath is very smooth, the top rivulet cannot form. With time, however, the smoothness of the surface reduces due to deposition of dust and the situation changes.

10.4.4 *Dampers and Cable surface Treatment to Control Vibrations* ***(Fig. 10.5)***

Mitigating countermeasures which are similar for both types of cables, and, consist of surface treatment to the outer HDPE tube, are necessarily required to be undertaken.

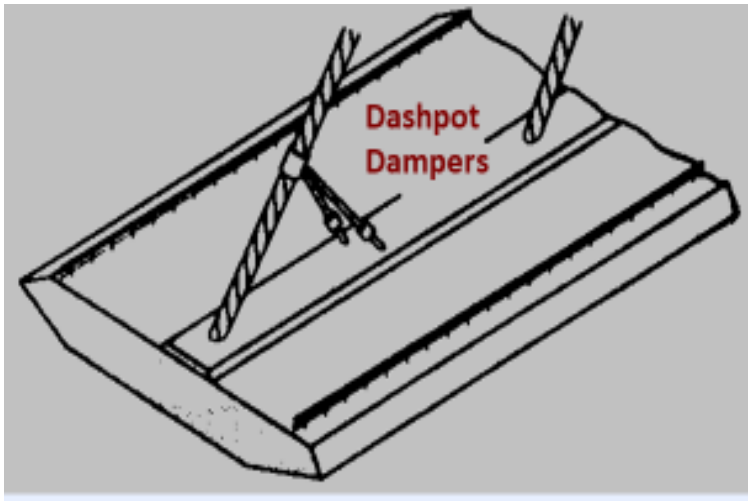
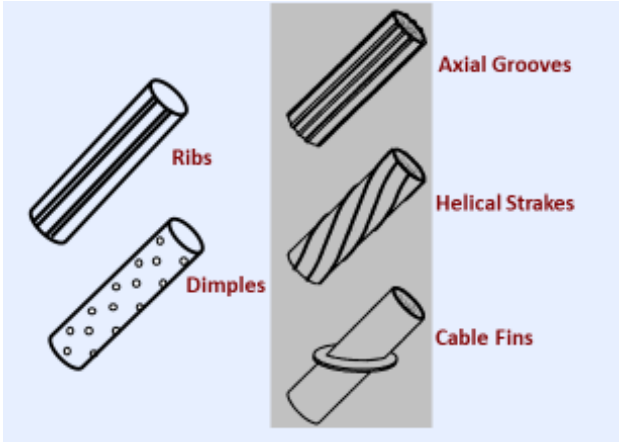
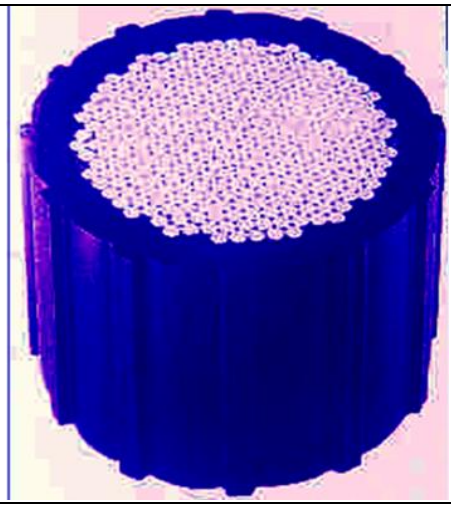


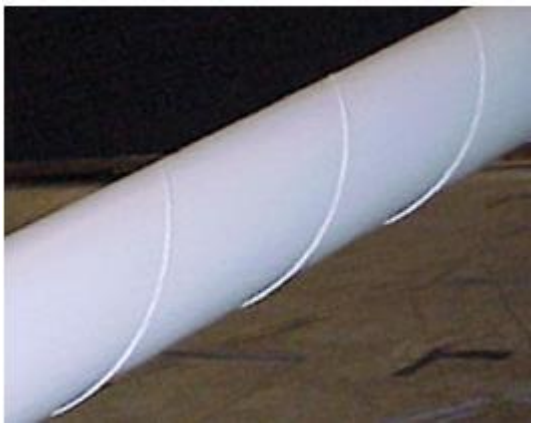
Fig. 10.5 Dampers installed on the stay



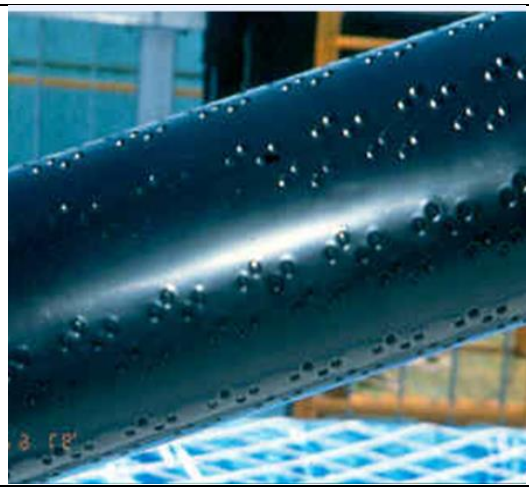
(a) Surface Treatment to Control Rain-Wind Vibration



(b) Axial Grooves on HDPE Tube



(c) Helical fillets on HDPE Pipe



(d) Indentations on HDPE Tube

Fig. 10.6 Stay Vibration control Measures

The preferred and proven treatments are either a system of double helical fillets running along the cable or a system of indentations or dimples in the surface. Protruding strips can also be provided. The double helical fillet treatment is usually employed for PSS cables while the dimples are used for NPWS cables. This different approach is due to suppliers' preference and the different manufacturing techniques. The purpose of the treatment is to prevent the formation of continuous rain rivulets running down the cables. These measures have proved to be effective in minimising the risk of large amplitude rain/wind induced vibrations. The actual shape and dimensions of the fillets and indentations must be optimised to minimise the effect on the drag of the cable. For both systems, wind tunnel tests have shown that with optimised dimensions only a very small increase in drag coefficient occurs.

Fig. 10.5 & **10.6** depicts the countermeasures for the type of vibrations described above in **Sections 10.4.2** & **10.4.3.** Besides the above measures, cross cables are employed, if it is required to change the vertical mode frequencies of the cables. Cross cables are often regarded as a last resort in suppressing vibrations as they, apart from requiring maintenance themselves, complicate inspection and maintenance of the main stay cables. However, it is recommended that the stay cables should be prepared for possible future installation of cross cables. For structures using LCS and NPWS cables no preparation is generally required for the cables due to the fact that they are compact and will be able to accept the required clamps for the cross cables. For PSS cables special collars are required at interfaces with the cross cables. **Fig. 10.7** is an example of such an application.



Fig. 10.7 Cross Cables (Normandy Bridge)

10.5 Bearing Plate, Recess Pipe Installation

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

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

The vertical and horizontal angles are especially critical, as offsets would create bending stresses (**Fig 10.8)** in the stay cable. The anchorage adjustable nuts or shims are in contact with the bearing plate. The bearing area must be perfectly flat to avoid stress concentrations and deformation of the anchorage socket or adjustable nut.

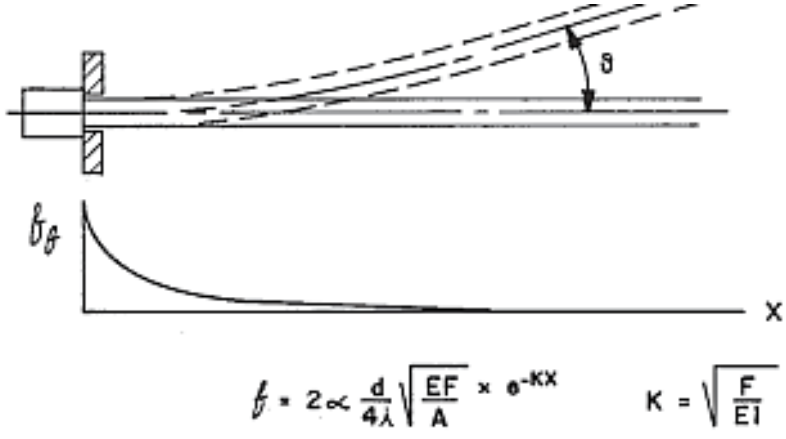


Fig. 10.8 Stay Bending

10.6 Stay Cable Pipe Installation

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

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

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

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

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

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

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

10.7 Installation & Stressing of Stay Cable Components

Stay Cable supplier instructions should preferably be followed strictly for the installation of the other stay cable system components. Guidelines for the same are enumerated in **Section 6.5.**

10.7.1 *Stressing of Strands*

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

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

(b) Multistrand jack stressing (gradient jack).

10.7.1.1 Single Stressing System Using Monostrand Jack (Isotension Method)

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

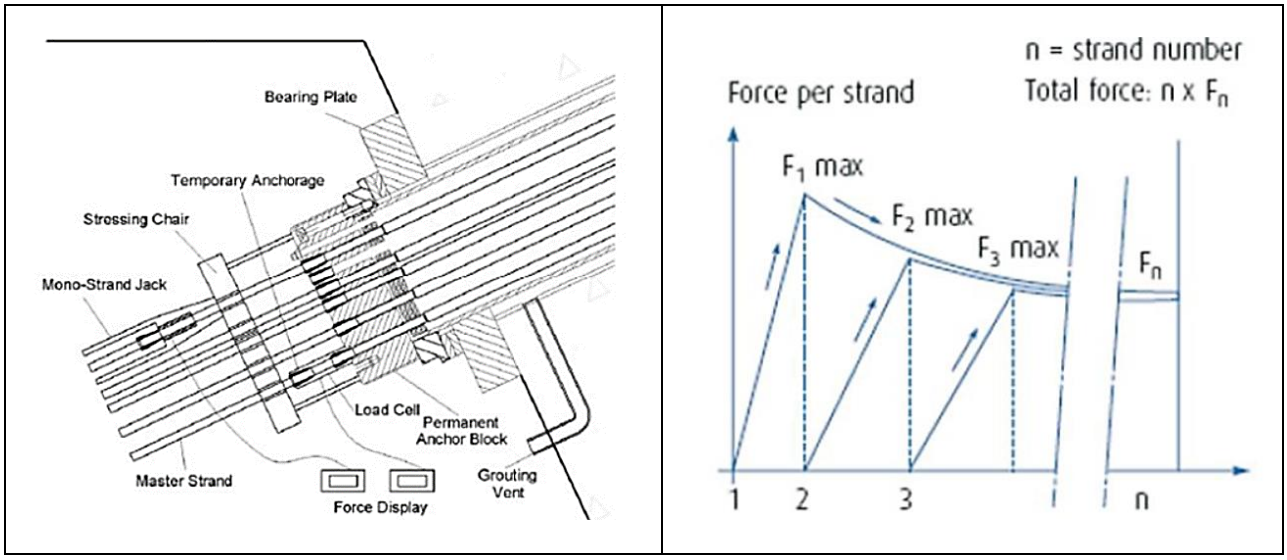


Fig. 10.9 Isotension Method and principle diagram

- The first strand ("the master strand") is fixed to one of the stay cable anchorages and its other end is threaded into the other anchorages and then cut and stressed to a pre-calculated force which has to be higher than the desired target strand force (when all strands of the stay cables are stressed). Such pre-calculated force needs to be arrived at based on analytical model taking account of flexibility of superstructure and tower.

- The wedge of the master strand is not yet engaged to the stay cable anchor block. The master wedge is instead wedged to a special single anchoring device provided with a load cell which gives the reading of the current master strand force.

- The second strand is then installed in similar manner, cut and stressed. The stressing is done as for the master strand, with a mono-strand jack equipped with a load cell identical to the one indicating the force of the master strand. As the second strand is stressed, the force in first strand decreases and the stressing operation is stopped when the readings of the two cell are identical. The second strand is then anchored/ wedged to the permanent anchor block. The two strands are equally stressed.

- The third strand is then installed and stressed until its force reaches the force of the master strand (which decreases jointly with the force of the second strand). The three are identically stressed.

- The same operation is repeated until the last strand of the stay cable is installed. The last reading of the stay is then to be compared with the target theoretical force (**Fig. 10.9**).

10.7.1.2 Multistrand Jack Stressing (Gradient Jack)

Final stay cable force may then be adjusted by the multi-strand jack, if necessary. Elongation of stay cable which can happen using multi-strand jack is very limited in the

stressing end anchorages design and its requirement must be defined by designer in the beginning of job to the anchorage supplier.

10.8 Cable Replacement

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

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

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

10.9 Protection from Fire and Vandalism.

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

However, the case may be different for an cable stay bridge with set of closely grouped stays or large number of trucks carrying hydrocarbon (near oil refineries) and covers a large numbers of stay cables. In such special condition, provisions need to be done so that temperature rise in the main tensile elements of the cable / anchorages must be retarded for a period, it would take to extinguish the fire.

Study of vehicle's base fire hazard may be required to carry out to asses required fire rating of cable stay system. Such study include:-

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

Special insulating material have been developed by stay cable supplier's for such purpose which enhances fire resistance to main tensile element (MTE) by protecting MTE to a level so that it can carry applied design load. Fire resistance qualification tests shall be required to be carried out to establish fire rating of cable stay system **[3]**. The 30-minute fire rating

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

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

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

10.10 Anchorages for Cables

The two ends of cable stay are called as anchorage zones. These consist of:

- Anchorage which transmits the tension of the cable stay to the attachment parts of the structure (deck, cross beams gussets, pylons etc..)

- Transition zone extends from the anchorage to the start of the free length of the cable stay. The transition zone comprises of deviators, transverse guide systems, external dampers and sealing system.

  The anchorage zone and the transition zone generally fulfil the following functions:-

- Transfer of Cable Stay Force- Anchorage consists of mechanical part (wedges) designed to secure the main tensile elements of the cable and transmit their forces to the attaching parts of structure.

- Filtering Out Angular Deviation- Anchorage may comprise cable guide systems in order to partially or totally prevent angular deviations of the cables extending to the anchorage head. Hence appropriate system be provided to limit or eliminate the effect of cable stay deviations at the anchorages head i.e to filter out changes in angle between the cable end and anchorages.

- Possibility of Adjustment- Anchorage has provisions for moving the anchorage head with respect to structure to provide cable stay tension adjustment and initial cable stay directional adjustment.

- Corrosion Protection and Water Tightness- The design of anchorage must allow the two barrier corrosion system of strands to be extended up to anchorage plate without any break. Certain layer of protection specified for free length can not be fully continuous from end to end of cable. This applies in the transition zones wherein sheathing of prestressing steel is removed inside the anchorage. Whenever one of the layers of protection is interrupted it must be replaced with an equivalent layer of protection.

- Provisions shall be made in anchorage to drain out water to clear out ingress through any stay barrier or stay-anchorage connection.

- Cable Replacement- The design of anchorage allows the replacement of complete stay cable as a whole (for LCS & NPWS cables) or in parts strand by strand (for PSS cables).

10.11 Anchoring / Passage of Stay Cables in Pylon Body

Two dispositions exist for the transition through the pylon head. The first cuts the continuity of the stay cable. In this case, there are 3 typical cases which are defined in (**Fig 10.10)** as cases A, B, C. The second keeps continuity of stay cable within the saddle, shown as case D .

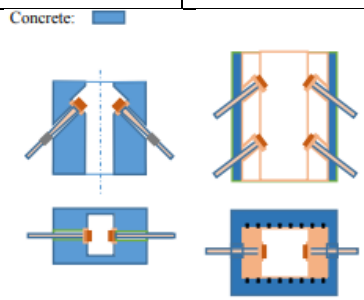
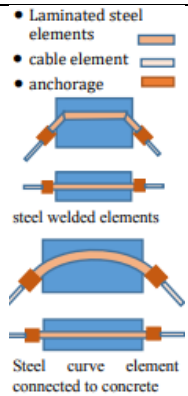
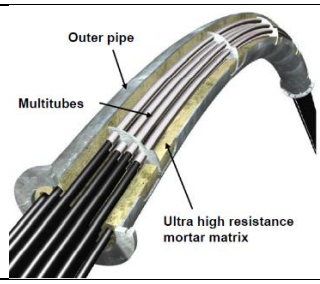
Type	A	B	C	D
Elevation Plan				 Courtesy-M/S Freyssinet International  Courtesy-M/s VSL
Comments	Cables anchored inside concrete pylon	Cables anchored inside composite element or in Mast head steel	Cables anchored in cast-in steel element	Individual holes for individual strands of cables passing over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

Fig. 10.10 Anchorage Configuration in Pylon

The A and B type of steel anchorage box structure at the head of pylon allows an easy maintenance, and as such these are extensively adopted in cable stay bridges. The working space which is necessary for installation of anchorages inside of pylon body requires internal shaft which increases pylon size. In order to take all equipment's up for stressing of stay cables and further inspection and maintenance at such anchorage location also requires access staircase, platforms etc through shaft inside pylon body.

Another option is where the stay cable is not continuous and is anchored on each side, on an external nose section where the cable can be anchored by means of standard anchorage. The C type of anchorages of stay cables can be done outside of pylon body. In such case, multiple numbers of curved profiled built up welded steel elements (with external nose element at its both end on which the stay cable are anchored) are pre-installed in the pylon body for all pairs of stay cables which are required to be anchored on either side of pylon. There is a internal steel link saddle for every pair of cable anchored on either side of pylon. For such case, it is always preferred that stressing of pair of cable stays (on either side of pylon) anchored at both end of internal steel link saddle shall be done simultaneously from its lower end (anchored to deck). This will induce direct tension in internal steel link saddle, thus creates an inward pressure to concrete body of pylon head. Any differential force in pair of stay cable anchored at both end internal steel link saddle get transferred to pylon concrete body through welded shear connector provided on outside surface of internal steel link saddle.

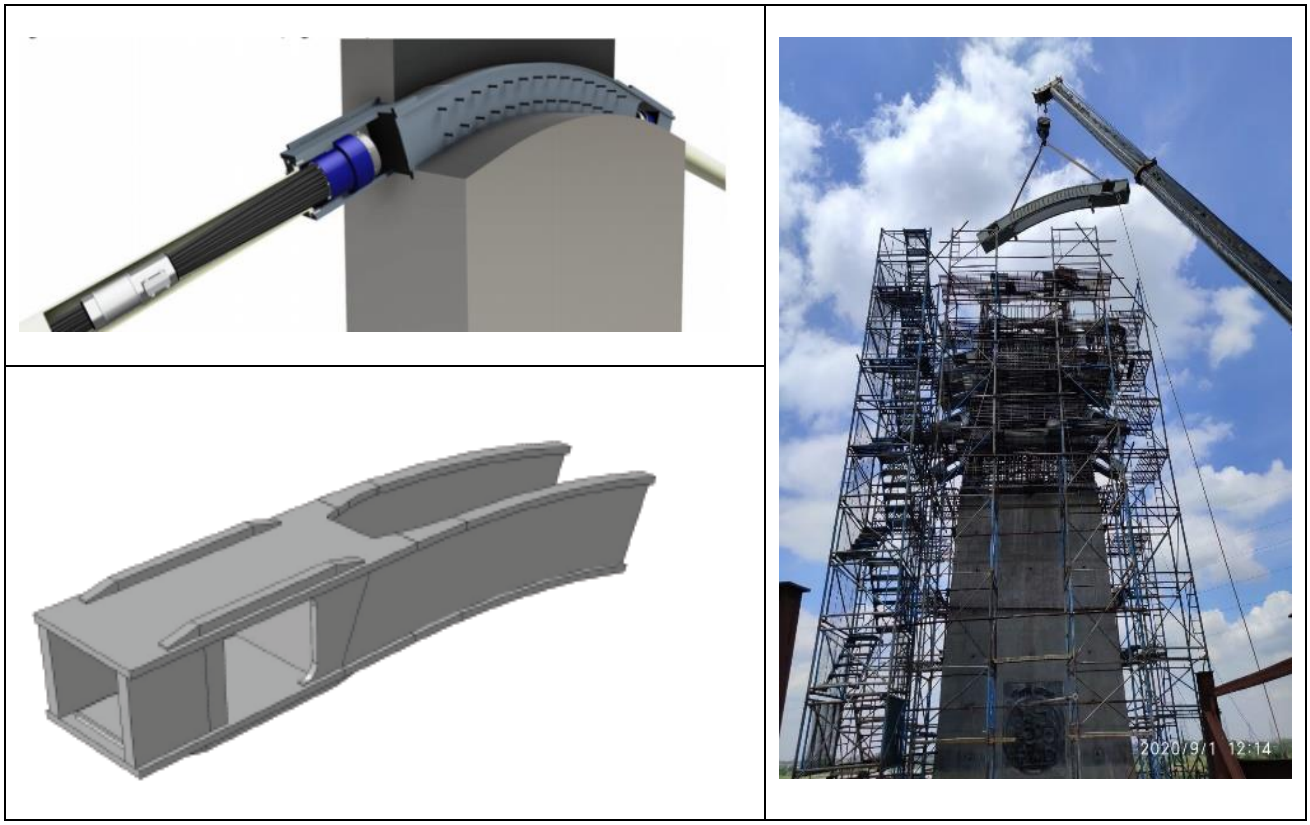


Fig. 10.11 Internal steel link type saddle

Lateral openings can be outfitted to allow an easy exchange of single strands or even whole cable if required. **Fig. 10.11**. In concrete pylon, such saddles have added structural advantages as: -

- Shear studs at the steel beams inside the pylon easily transfer differential loads within the concrete although a proper check of elongation vs shear stud loadings needs to be performed.

- Friction, bond stresses and mechanical wear due to strand deviation are avoided.
- An unbalanced installation sequence as well as the use of two different cable sizes at one anchor box becomes possible.
- The left and right side cables of one box can be differently inclined, and there are no limits in the respective deviation radii.
- The design of the anchor box is fully in accordance with conventional steel construction standards with element always in tension.
- If any inspection and maintenance of the corrosion protection at the cable anchorage is required, it can be performed easily from outside the pylon structure.
- Possibility to design a slender and elegant pylon
- No necessity of an access inside the pylon

In some cases, instead of anchoring the cable in pylon, cable stays are allowed to continuously pass thorough pylon by providing a saddle (D type in **Fig. 10.9**). There is a saddle per cable. These allow the continuity of tensile elements without gap and limit the sizes of the pylons, and, provide the anchorage for the cable while avoiding their slippage. The advantages provided by saddles as reported generally are:

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

However, saddle seldomly used in cable stayed bridges as inclination of stay cable with horizontal is too large and difficult to negotiate. However, if adopted, it should be preferred option for bridges in non-seismic zones. Saddles, which are solely depend on friction to block differential forces in stay cables on either side of saddle shall not be used in high seismic zones. If adopted, detailed structural investigation of the bridge shall be carried out using site specific response spectrum or time history analysis using spectrum compatible time histories.

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

If at all, saddles (with individual hole for individual strand only) are proposed by designers. In that case following shall be ensured:

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

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

For Saddle system, requirement of initial testing and suitability testing as given in **Section 10.3.2 & 10.3.3** respectively shall be followed.

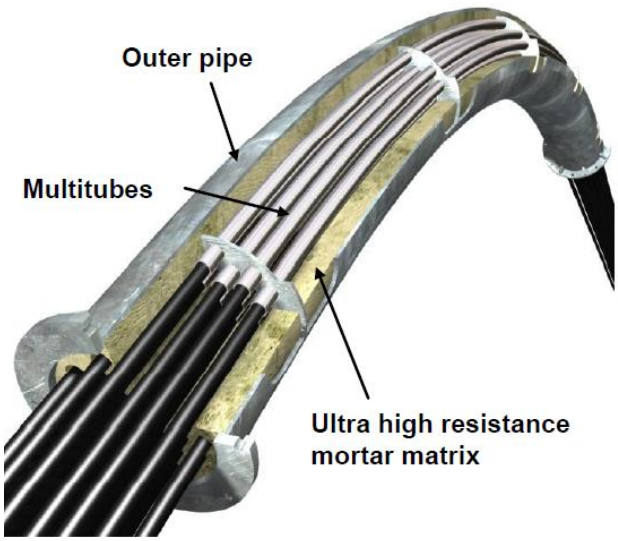


Fig. 10.12 Multi-Monotube Saddle

Conventionally, multi-mono tubes saddle (**Fig. 10.12)** are used with closely spaced diaphragm and space around tubes is filled with high strength mortar. Each strand of cable is allowed to pass through each curved profile tube. However, in such cases, strands are sheathed with specially designed resin or epoxy coated so that fretting corrosion of strand does not happen inside the tube.

Structural Saddles (**Fig.10.11)** in which main tensile elements are coupled at both ends shall be designed in accordance with standard applicable **[5]** and tested in accordance with conventional stay-anchorage system requirements.

10.12 Levels of Protection for Preventing Corrosion of Stays.

10.12.1 *Corrosion Protection of Stays*

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

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

10.12.1.1 Parallel Strand System (PSS) Cable

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

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

10.12.1.2 New Parallel Wire System (NPWS) Cable

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

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

10.12.1.3 Locked Coil System (LCS) Cable

For LCS cables the second barrier is either in the form of a paint system applied to the complete assembly or of an extruded polyethylene sheathing around the wire bundle. While an extruded sheathing might provide a slightly better protection than painting and might be preferred for aesthetic reasons, repair and inspection of a painted cable is easier and this is therefore the most used option. The interlocking Z-shaped outer wires form in combination with the self-compacting effect from the spiral arrangement a tight surface preventing moisture to penetrate and travel along the inside of the cable in case the outer barrier is damaged.

10.13 Dampers for Stay Cables.

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

The risk of vibration increases with cable stay length. Short cable stays (generally less than 80 m) generally involve no risk, other than parametric resonance in the case of a particularly unstable structure. Hence, such damping devices are generally needed, if the cable length is more than 80 m.

10.13.1 *Internal Dampers* ***(Fig. 10.13)***

These are of ring shaped and installed inside guide tube which in turn fixed to formwork tube. It means, they are always installed near the anchorage at lower end of cable stay. Internal dampers use to dissipate the energy either by the distortion of a dissipating material or friction of two interfaces. Various suppliers have different form of internal dampers to achieve their objective.

In case of internal elastomeric damper, dissipating material can be specially formulated rubber.

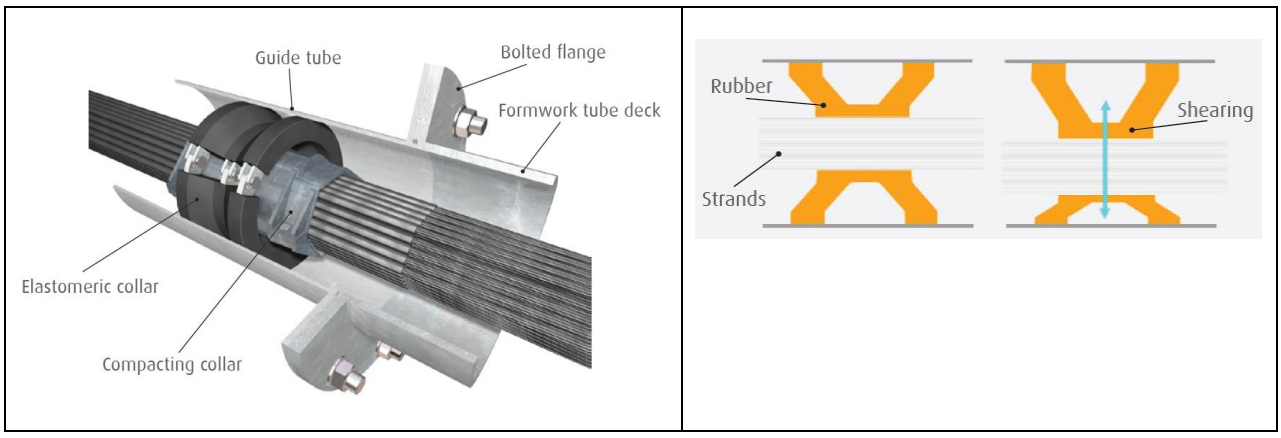


Fig. 10.13 Internal Elastomeric Dampers (Courtesy – Freyssinet Dampers)

In case of hydraulic dampers/viscous damper **(Fig 10.14),** resistance is induced by the rapid passage of a viscous fluid through a narrow opening. The resistance can dissipate a large amount of energy leading to the damping of cable.



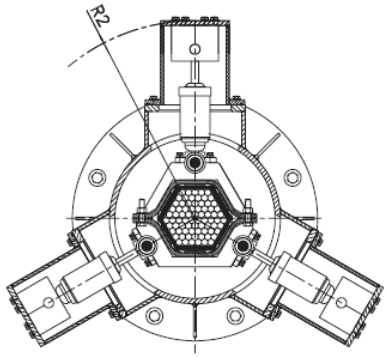


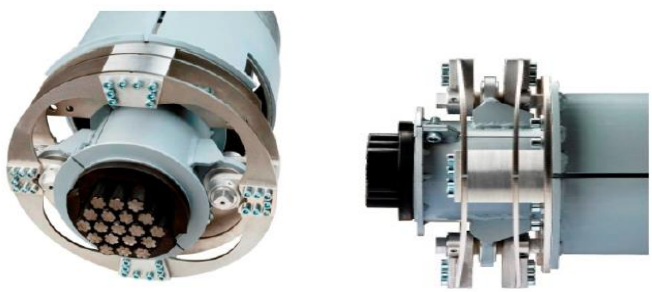
Fig. 10.14 Internal Radial (Viscous) Damper (Courtesy – Freyssinet Dampers)

In case of friction dampers **(Fig 10.15)**, sliding interfaces would dissipate the maximum of mechanical energy while keeping the threshold amplitude below a specified acceptable limit for the structure in terms of fatigue or user comfort.

Hence, friction Dampers is the efficient solutions for critical cases on long stay cables or where there is a risk of parametric excitation. Its non-linear damping characteristics are particular suitable to prevent cable vibrations independent of the vibration mode. The friction behaviour prevents the dampers from being activated at small amplitudes which are considered not to be critical for the cable performance. Once a certain amplitude is exceeded, the damper provides immediately an unrivalled amount of damping & preventing the build up of any large amplitude.



(Courtesy – BBR Dampers)



(Courtesy – VSL Dampers)

Fig. 10.15 Friction Dampers

10.13.2 *External Dampers* ***(Fig. 10.16)***

For very long cables external dampers are more efficient than internal one. These are of two types:

Fixed External type: These are hydraulic devices exerting a transverse damping force on the cable near the anchorages. They are installed in pairs directly on the stay.

Pendular External type: It also uses a piston damper with a pendular lever system free to move around rod hinged on a fixed support.

	
Fixed type (with pair of piston)	**Pendular Type**

Fig. 10.16 External Damper (Courtesy – Freyssinet Dampers)

10.13.3 *Installation & Maintenance of Dampers*

Dampers are usually installed, once cables are structurally active and carry the permanent and superimposed loads of the structure. After installations, factors such as service load, traffic, wind and temperature modify the geometry of the entire structure and consequently induce relative rotations between the structure and the stay cables, which result in transversal and longitudinal movement at the damper locations. These movements are

often larger than those imposed on the damper and the anchorages by possible cable vibrations. To ensure good damping performance, durability and safety, the damper, stay cables and anchorages must be analysed, designed and detailed as a whole. Consequently, both the stay cable and damping device should be provided by stay cable system supplier.

The damper and elasticity properties of a damper must be adjustable and adaptable to the each cable stay on a case to case basis.

Dampers undergo lot of wear and requires regular monitoring and, maintenance. Detailing of the damper shall be such that accessibility, adjustability & replaceability of either part of damper or as whole can be easily done.

10.14 Protection of Cable stays From Ice Accretion with De-Icing & Anti-Icing Systems

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

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

Pieces of accreted ice or snow fall from bridge members occurs

- when their weight increases beyond a value
- when temperature of outer surface of structural rises above $0^{o}C$, melting occurs at the surface-ice interface.

Block of Ice or Snow falling from cable stay / pylons may result in traffic accidents which can cause direct damages to passing vehicles and human safety at risk. Also this can result in fear / anxiety amongst drivers. Consequently, lack of successful de-icing or anti-icing measures may result in bridge closure which leads to traffic hindrance that can in turn lead to severe financial losses. Denmark's Great Belt Bridge was closed for 12 hours per year during falling of ice. Oresund Bridge, Severn Bridges of UK and Uddevalla Bridge of Spain have suffered similar experiences.

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

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

10.14.1 *Mechanical Method* ***[4]***

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

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

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

10.14.2 *Thermal Method* ***[4]***

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

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

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

10.14.3 *Passive Method* ***[4]***

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

- Thermal absorbent coating which is effective only when sufficient radiation is available.
- Hydrophobic coating with surface tension and adhesion which is only effective in wet snow conditions.
- Solid icephobic coatings but still have ice-adhesion forces ten times greater than what is necessary for ice to detach under gravity or wind loading.

- Viscous products and anti-icing greases but have limited duration of protection as they loses their efficiency under the effect of precipitation.

10.14.4 *Conclusion*

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

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

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

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

In India, nothing much has been done or in operation. It is high time to adopt EIDI or PETD system on cable supported bridges in snow clad areas.

10.15 Durability Design of Cable Stay Bridge

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

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

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

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

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

Based on ease of accessibility and condition of its replaceability of mild steel components of anchorages and saddle, cable stay system supplier shall present the corrosion protection approach of his cable stay system in accordance with **Table 10.1**.

Table.10.1 Corrosion protection philosophy for mild steel components of Anchorages & Saddle [2]

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

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

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

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

CHAPTER - 11

BEARINGS, EXPANSION JOINTS AND SEISMIC PROTECTION DEVICES

11 Introduction

Cable-stayed bridges like suspension bridges are highly stressed structures coupled with fluctuating stresses due to dynamic forces (wind & seismic), thermal expansion / contraction, shrinkage & creep, etc. Such bridge structures undergo a large movement and rotation along-with high fluctuating load situations inducing demand for a structural support and protection system capable of catering to such forces and movements. By and far bearings and expansion joints for cable stayed bridges shall be designed as per the relevant IRC codes and guidelines. Some of the special devices and relevant precautions relevant to large span bridges are indicated in this chapter.

Given the length of continuous superstructure and the bearing arrangement, movement at bearing and expansion joint levels can be as large as up to 2 m. The bearing rotations are also often large, in the range of 0.03 – 0.15 radians. Hence, a highly durable & robust design & manufacturing of these devices is necessary.

11.1 Basic Concept of provision of Bearings and Expansion Joints in Cable Stayed Bridges.

This Chapter illustrates the utilization of bearings and expansion joints for long span cable stayed bridges with examples. **Fig. 11.1** shows a 350 m main span cable stayed bridge over river Chambal. Economical configuration of CSBs occurs when the main span exceeds 200 m which has been explained in **Chapter 4**.

Chapter 4 discusses the structural action of cable stayed bridges. It illustrates that in the service condition bearings are usually not provided to support vertical loads (DL, SIDL, LL) of the deck at the pylon location. However, since the side spans are usually about 50% of the main span, the end supports are subject to uplift which has to be countered by providing kentledge in some form. The space inside hollow box girder can used quite conveniently for the purpose of filling concrete, etc. Other possibilities are to increase the weight of part of the side span and also to use the weight of the approach span as a counterweight, as was done in the case of Naini Bridge **Fig. 11.2** as well as Tatara bridge, Japan **Fig. 11.3**. Another solution discussed in **Chapter 4** is to continue the deck beyond the main span into the shorter spans of the approach structure **Fig.11.1** (Chambal Bridge, Kota). In effect, it means that the end pier as well as other piers, when provided, must always be provided with bearings, sometimes to cater to uplift / tension. The bearings require capacity of large movements in the longitudinal direction even though the vertical loads may be small.

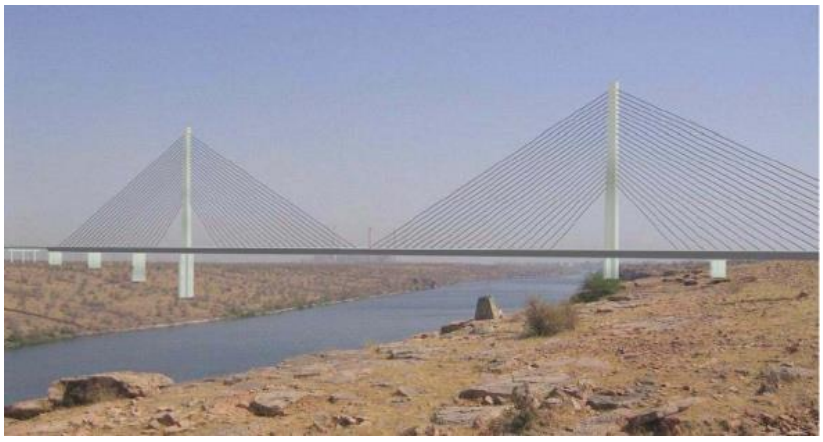


Fig. 11.1 Kota Bridge over Chambal, Rajasthan

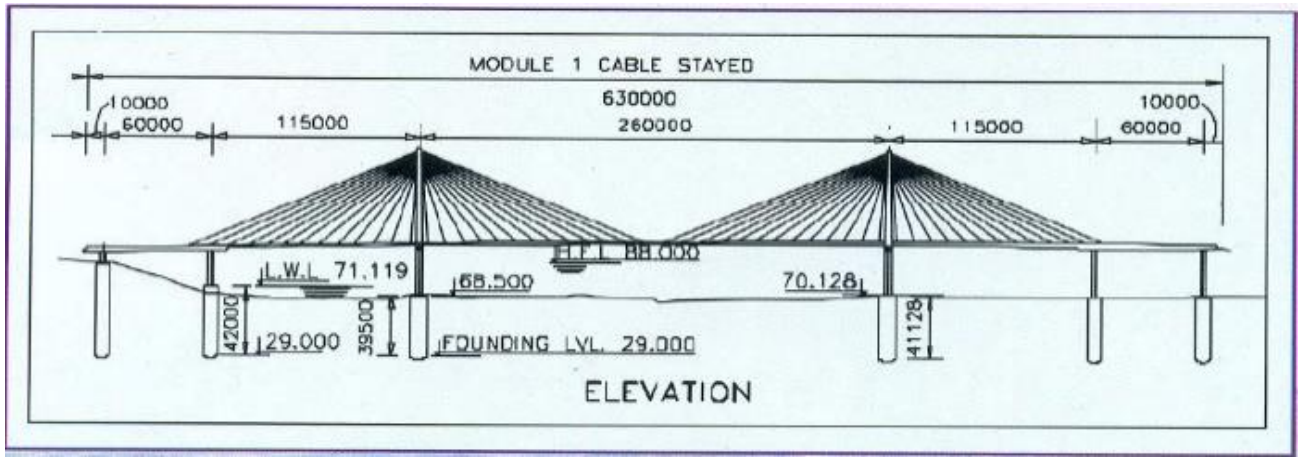


Fig. 11.2 Naini Bridge

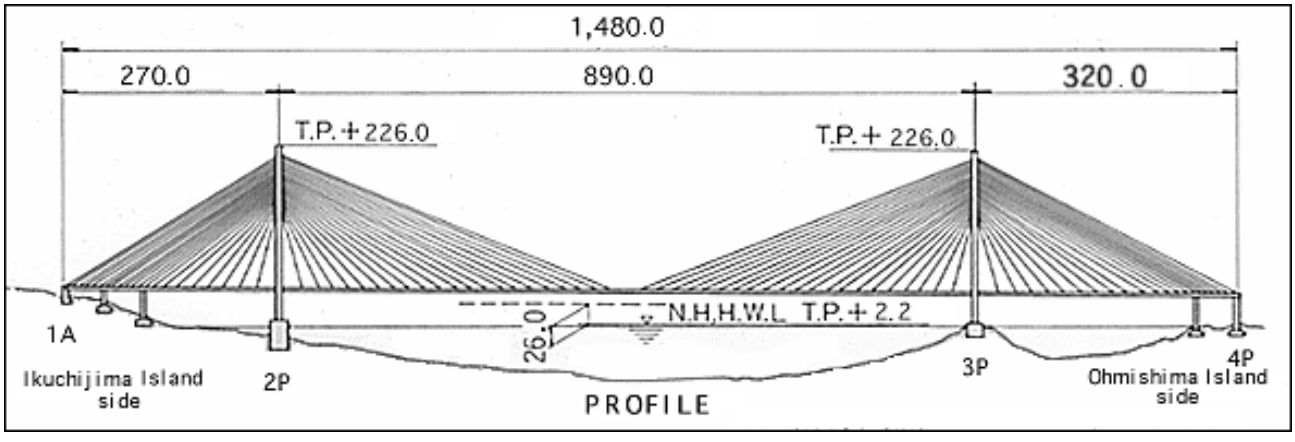


Fig. 11.3 Tatara Bridge, Japan

Since, the lateral forces due to wind and earthquake on the deck in the transverse direction of the bridge require to be taken care of before being transferred to the foundations, lateral restraints at the supports in **Fig. 11.4 & 11.5** (at all four or at least at two of them) in the form of suitably designed bearings in vertical plane are usually provided.

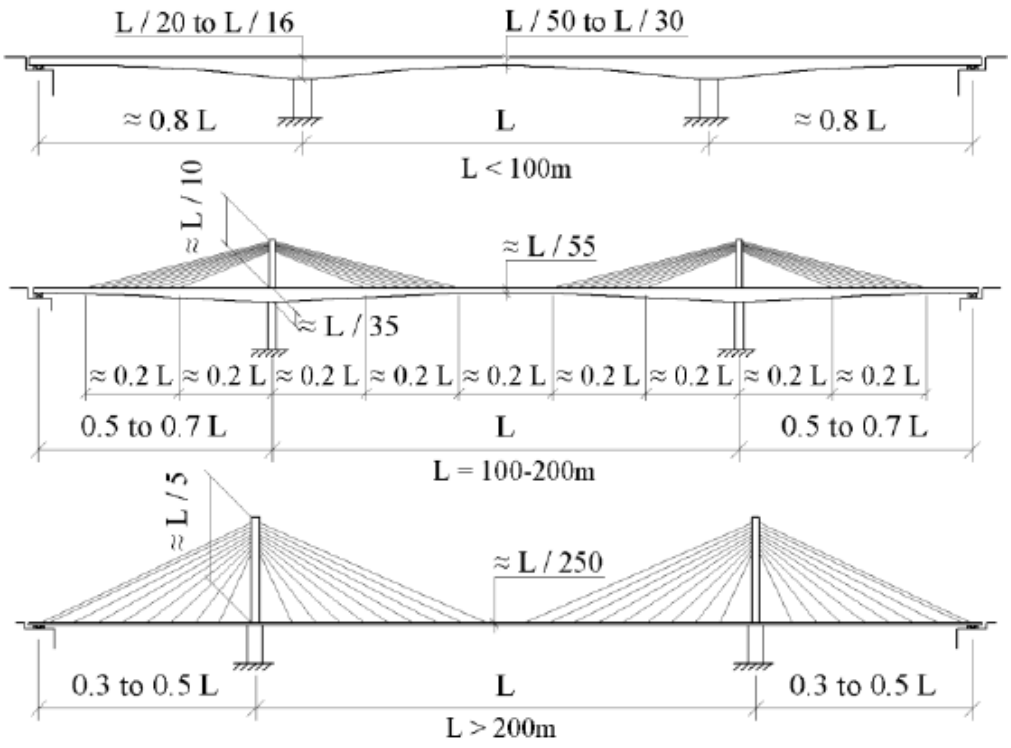


Fig. 11.4 Three Span Configuration

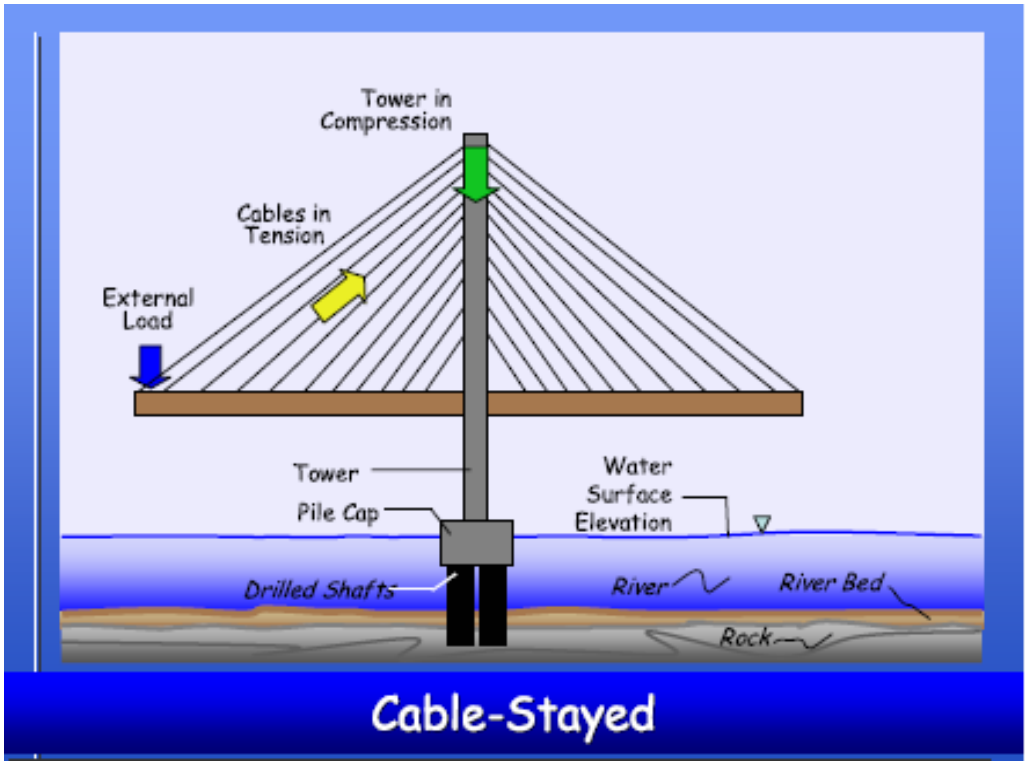


Fig. 11.5 Cable Stayed Bridge During Construction Stages

The Expansion Joints become necessary to enable the long continuous structure to breathe due to large thermal expansions & contractions, creep and shrinkage effects and also to cater to movements of the deck due to braking, earthquake and wind effects. Even for a small cable stayed bridge with main span of 200 m and side spans of say 100 m each, the distance between the expansion joints would be as large as 400 m. Movement from such large interspacing between Expansion Joint may be too large for a single strip seal or other regular types of expansion joints that are suitable for smaller movements and therefore Modular Strip Seal Expansion Joints or other suitable types become necessary. In high seismic areas certain special features in the expansion joints become necessary. These will be discussed later in this chapter.

Right selection of Expansion Joints, Bearings, Seismic Devices and vibration control devices becomes extremely important in case of Cable stayed Bridges where significant daily service movements, inducing fatigue and substantial accumulated sliding path occurs.

In this chapter the essential requirements specific to behavioural outcome of Cable stayed Bridges that translates to performance requirements of Expansion Joints, Bearings and Seismic Devices are discussed as below. Vibration control devices are discussed in other relevant chapters.

- **Wind effects:** Cable Vibrations as well as Vertical deck movements with fatigue effects. Increased stress / load levels, Accumulated sliding path in Bearings and Joints. Large wind forces into Bearings (lateral, up-lift, etc.)

- **Traffic effects:** Cable vibrations, Vertical & lateral deck movements and rotations with fatigue effects. Variation stress / load levels in the Bearings and Expansion Joints. Large accumulated sliding path in Bearings and Expansion Joints.

- **Seismic effects:** Anchoring forces to be transmitted through Bearings. Also, the very high movements.

In the following Sections, Bearings and Expansion Joints that are suitable for Cable Stayed Bridges are discussed.

11.2 Bearings

Bridge bearings must be able to transfer the loads & forces, hold the deck in position and allow expansion & contraction.

Caused by the dynamic actions and characteristics of Cable Stayed bridges, the Bearings system function in the bridge may require freedom in up to all 6 degrees. Cable Stayed Bridges impose repetitive rotation and translational movement at higher frequency, magnitude and velocity.

The selection of the appropriate bearing type shall be such as to fulfil the durability and service life demand of the Structure. This shall be fulfilled considering the accumulated movements, rotation cycles combined with wearing and fatigue effects. Bearing design shall cover SLS, ULS and simultaneous wind / seismic load cases whichever is governing

without any compromises on their performance. Various types of bearings applicable to cable stayed bridges are discussed below.

11.2.1 *Elastomeric Bearings*

Reinforced Elastomeric Bearings are simple yet versatile. These consist of steel laminates sandwiched between the Elastomer Layers. While the elastomer provides the ability to take rotation and translation by deflection, the ability to bear & transmit the Vertical Loads and Forces comes from the Steel laminates reinforcement.

Elastomeric Bearings follow simple configuration for transmitting the vertical loads, translation and rotation. The Elastomeric Bearings have moderate life span due to rubber material characteristics which is subjected to ageing under open environment as well as the wear due to regular movement and rotation. However, these bearings have the advantage of easy inspection and replacement, if required.

Elastomeric Bearings require larger space as they have low permissible design Stress as compared to mechanical Bearing systems. The restoring moment from the Elastomeric Bearings under peak rotation and movement conditions is required to be carefully analyzed while designing the associate components for e.g. the sliding interface combined with the Bearings. Elastomeric Bearings being vulnerable to ageing, ozone and fire may also need additional precautions. Experiences indicate that exceptionally large size elastomeric bearings require additional construction for their easy & safe lifting and handling without damaging the soft elastomer surface of the bearings.

Elastomeric Bearings may require special consideration for application in Cable Stayed bridges, as they are subject to considerable load fluctuations. Depending on their limitation to transfer high horizontal forces (such as those due to wind & earthquake, braking, etc) as well as rotations and translations, operating temperature etc, Floating Bearing configuration of elastomeric bearings may not be feasible. The arrangement of restraints or guide bearings in addition to Floating Elastomeric Bearings may provide a meaningful, long-lasting solution.

11.2.1.1 Anchored Elastomeric Bearings

The Elastomeric Bearings, generally, do not require positive Anchorages for fixing under normal bridge applications. However, for applications under dynamic loading conditions such as those in Cable Stayed Bridges, where the Elastomeric Bearing is susceptible to slippage out of position on account of extreme load fluctuations due to live load, wind and seismic, it may be appropriate to provide Elastomeric Bearings with positive Anchoring Devices. This would otherwise lead to slippage and dislocation of the bearings.

Anchoring of the Bearing can be effected in a most simple and cost-effective manner by means of an anti-skid plate (**Fig. 11.6)**, which is connected as an outer end plate to the bearing by effective vulcanization and fixed to the adjacent Structure by means of cementitious grout or epoxy bonding. However, for applications where this simple construction is not adequate and higher forces and extreme load fluctuations and vibrations are anticipated, more robust system of positive anchoring which includes top and bottom

vulcanized plates and fixing to the adjacent structure by means of Bolts and Dowels shall be considered **Fig 11.7**.

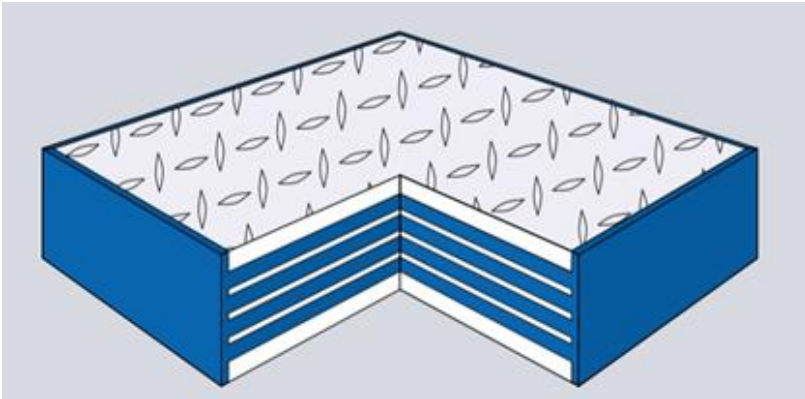


Fig. 11.6 Elastomeric Bearing with Anti-skid plate

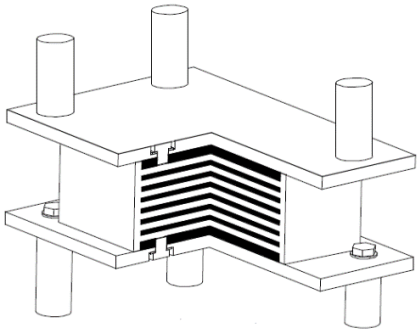


Fig. 11.7 Elastomeric Bearing with positive Anchorage

11.2.1.2 Lateral Restrained / Side Stopper Elastomeric Bearings

These Bearings are installed in Vertical position between the projections from Substructure and Superstructure (**Fig. 11.8 & 11.9)**. They provide resistance to impact & horizontal forces to the structural members. Vertical Stopper Bearings can be of Fixed type as well as Sliding Type depending on the movement requirement at the location they are installed.

These Bearings are provided with vulcanized steel plates on one side for accommodating the positive anchoring devices.

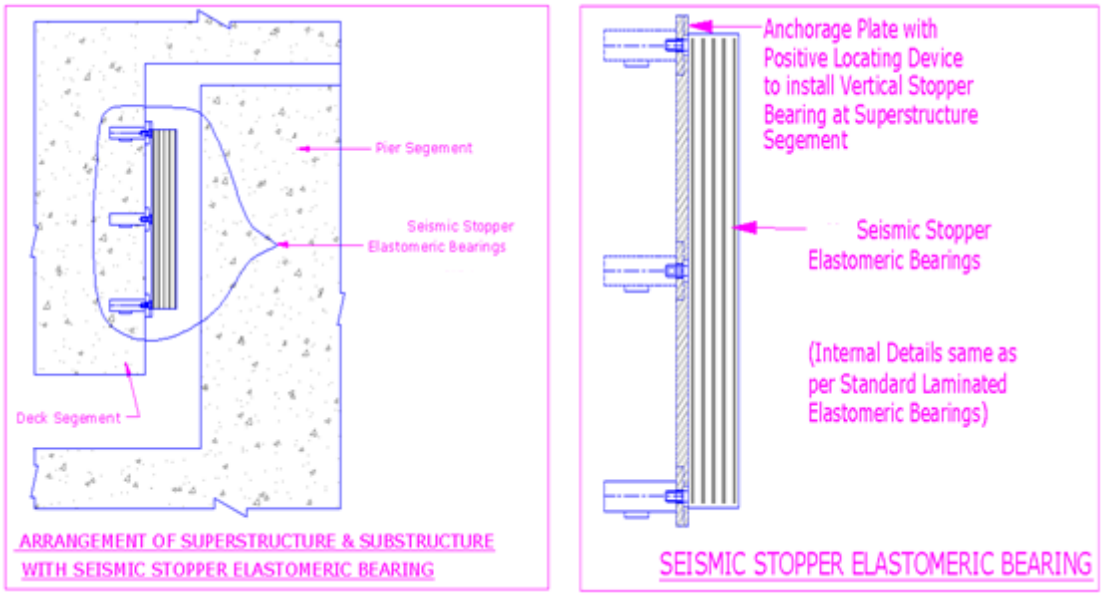


Fig. 11.8 Seismic Stopper Elastomeric Bearing with positive Anchorage

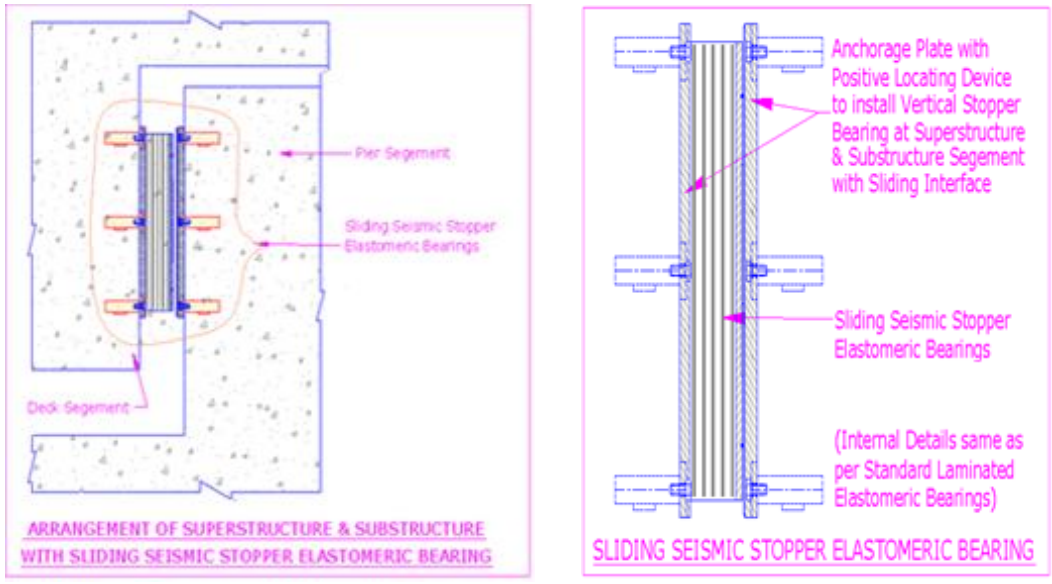


Fig. 11.9 Sliding Type Seismic Stopper Elastomeric Bearing with positive Anchorage

11.2.2 *POT Bearings*

POT Bearings can offer freedom up to 4 degrees of movement and rotation. The pot bearing consists of a confined disk of elastomer within a short steel cylinder called POT. Loading is applied via a close-fitting steel piston. This puts the elastomer under high pressure, making it behave like a liquid, permitting rotation in any direction with very little resistance. The design allows high vertical loads. Via pot wall and piston large horizontal forces can be transmitted. (**Fig.11.10 & 11.11)**. In the **Fig. 11.10**, (1) is the seal, (2) is PTFE and (3) is POT – Cylinder interface.

Fixed pot bearing offers transmission of Vertical Loads and Horizontal force while bearings equipped with a PTFE sliding surfaces allow free movable and guided design. Due to comparatively larger translational movements, importance must be given to testing for enhanced sliding path of the sliding components.

The performance and service life of Pot Bearings mainly depends on the effectiveness of the internal seal, intended to prevent the extrusion of elastomer that is under high pressure at the contact between the pot & piston.

The performance life of the POT Bearings largely depends on the service life span of the elastomer seal vis-a-vis accumulated rotation requirement at the bearing location which largely depends on the dynamic actions like live load deflection, temperature and wind induced rotation and deformations. Gapping effect at the sliding interface due to eccentric loading & rotation which is caused by elastomer stiffness is important for design consideration. Pot Bearings have limitation in satisfying large rotations. Specified limit of overall rotation is 0.03 radians and live load rotation limitation is 0.0025 radians unidirectional (for end support) and 0.005 radians (when installed at mid support of continuous span). Thus, due consideration is needed for Pot Bearings application in Cable Stayed Bridges where frequent & large magnitude rotation and dynamic movement situations are anticipated otherwise Pot Bearings may show signs of early maintenance and repair / replacement due to wear of Sealing Rings & Elastomer Pads.

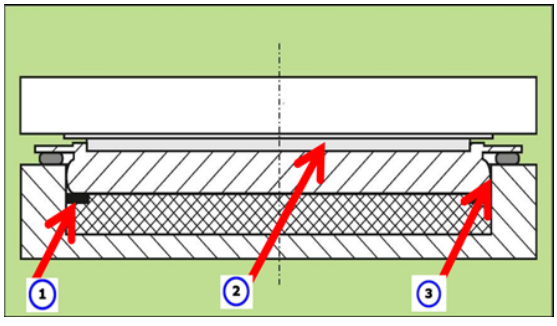


Fig. 11.10 POT Bearing Configuration

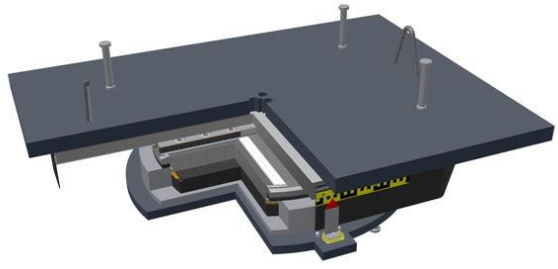


Fig. 11.11 Isometric View of POT Bearing

Further, due to the typical construction of POT Bearings adequate protection is needed at Pot-piston interface to prevent moisture / water ingress and standing water inside the POT due to gravity effects, to avoid corrosion. This corrosion between piston and POT would not only block the Bearing function but can also accelerate wear & tear of the seal.

11.2.3 *Spherical Bearings*

In this design, a spherical calotte corresponding to a concave lower part allows rotation about any axis (**Fig. 11.12, 11.13 & 11.14)**. A flat sliding interface allows transition. The spherical bearing combined with PTFE and other improved sliding material has proven itself to function well. The bearing can be inspected easily in the installed state. The use of new modified sliding material not only enhances the accumulated sliding path capacity but also increased load carrying capacity over a smaller footprint as well as the ability to better accommodate the load variations that commonly exist in case of Cable Stayed Bridges.

The use of such modified sliding materials adds to the service life of the bearings and economy, especially, under large loads.

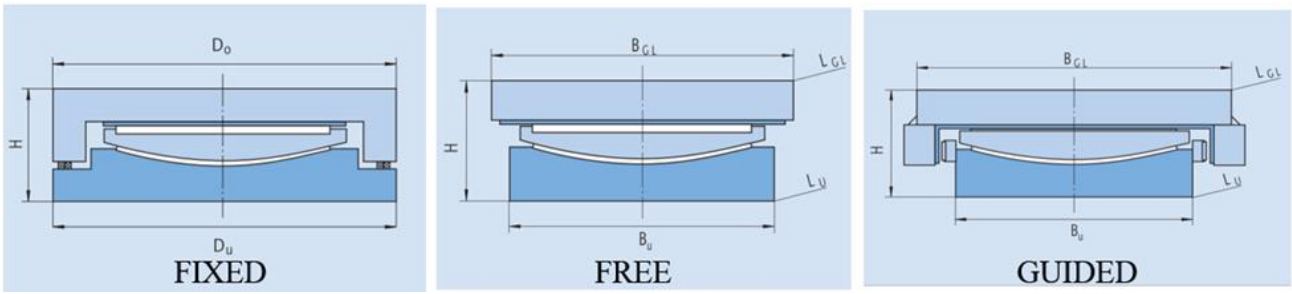


Fig. 11.12 Typical Configurations of Spherical Bearings

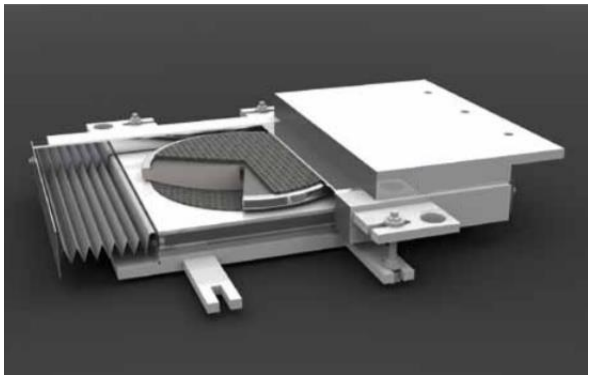


Fig. 11.13 Cut Section of Spherical Bearing



Fig. 11.14 Concave - Convex Rotational Interfaces

Due to comparatively large translational movements and high frequency dynamic rotation, importance must be given to testing for enhanced sliding path of the Sliding components. Also, care is required during their installation to avoid accidental disturbance to the level, alignment of the bearing and that of the superstructure resting onto the bearing prior to completion of superstructure skeleton. This is because Spherical Bearings unlike the Elastomeric and Pot Bearings can deflect easily with minimal amount of force.

11.2.4 *Anti-Uplift Bearings*

In Cable Stayed Bridges, often at the location where Back Stay cables are anchored to the deck or structural member holding the stays, bearing arrangement often requires anti-uplift consideration to resist occasional or permanent tension forces. Also, the combinations of varying loads and forces due to traffic, wind or seismic condition can create the requirement of uplift or tension forces which are to be resisted by such devices. **Fig. 11.15** depicts a typical Anti-uplift Spherical bearing.

It is worthwhile to mention here that options of handling the condition of Tension or Uplift forces by Structural Design consideration or by implementing special arrangements like Tension Bars or holding down cables and bolts are also worthy appropriate solutions. However, it is possible to design the bearings to handle such special conditions by themselves.

For the designing of Anti-uplift Bearings, due importance shall be given to whether the condition of uplift arising is only in ULS / Seismic or is in SLS also, i.e. day to day Service Condition. There is a need to differentiate between frequent uplift / tension or occasional Uplift as this affects the Fatigue Design requirements which are based, not only on the expected magnitude of load but also the corresponding number cycles during the intended Service Life period. Fatigue Design Considerations shall apply when designing the bearings for frequent Service Uplift conditions. Established literature and approved documents may be followed for the fatigue design.

Challenges in Design and Production:

i. The uplift forces including applicable horizontal movements shall be carried safely by the Bearing in any service and ultimate load condittion.

ii. The Bearing system shall not develop significant Vertical Play / Gap in uplift condition which may otherwise cause fatigue effects due to impact loading and rattling of the components in load reversal conditions.

iii. All components of the Bearing must be capable of carrying adaequate number of load cycles without fatique.

iv. Lifetime, friction and restoring moments shall not be restricted by the uplift design.

v. Since the standard Bearing design rules may not be adequate for design of Uplift Bearings, special design considerations and FEA analysis may be used for design verification.

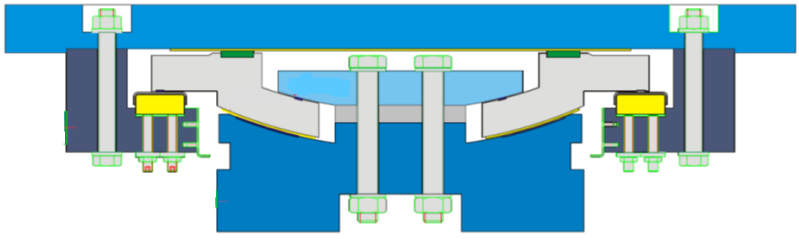


Fig.11.15 Typical Anti-uplift Spherical Bearing

11.2.5 *Tension Bearings*

Unlike the Anti-uplift Bearings which have to resist occasional uplift under load reversal conditions, the Tension Bearings are those which are deployed at the Locations where a permanent tension force exists simultaneous to requirement of freedom for translation and rotation. Such applications are common at the location of back stay cables anchoring to deck or at anchor locations in a cable stayed Bridge where permanent tension forces might occur. This requirement can be dealt with by providing long pre-stressed anchor bars freely suspended and fixed with spherical end plates and nuts or by pendulum system with spherical hinge connection at both ends for providing rotation as well as limited sliding freedom. For enhanced displacement capability, the design of pendulum Bearings can be modified with sliding interface while ensuring no separation at sliding surface under all

conditions of loads and movement. It is important to ensure the transfer of huge tension forces deep within the substructure by means of long anchor bars thereby satisfying the local pull-out failure condition, **Fig. 11.16 (a)**.

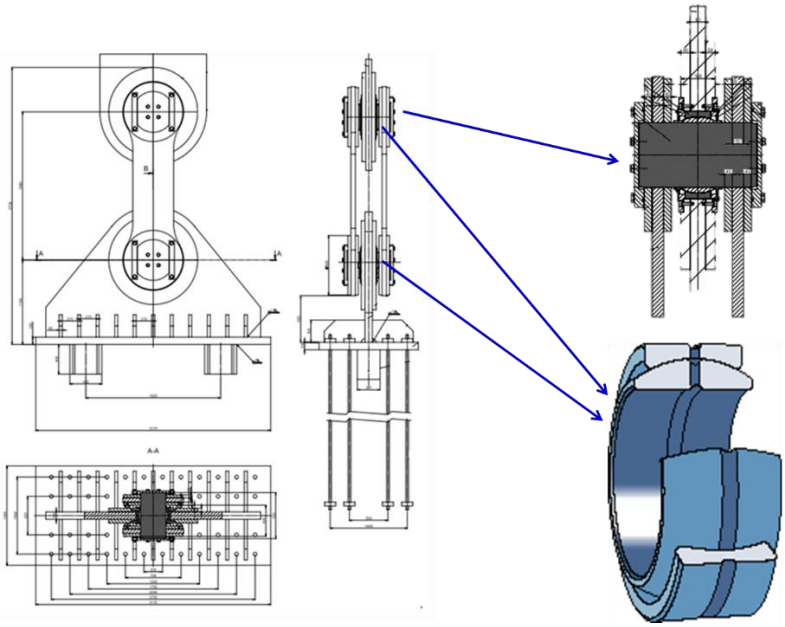


Fig. 11.16 (a) Typical Sketch of Pendulum Tension Bearing with Deep Penetrating Anchors

Signature Bridge in Delhi has employed such Pendulum Tension bearings. **Fig 11.16 (b)** depicts the bearing for tensions coupled with seismic forces. Design of Such Bearings also requires sufficient experience and considerations similar to Anti-uplift Bearings. Approved specialist literature may be followed for the purpose.



Fig. 11.16 (b) Pendulum Tension Bearing used in Signature Bridge, Delhi

11.3 Expansion Joints

Movement Joints commonly called as Expansion Joints are proprietary products which are classified into different types, primarily based on their movement capacities. As per the IRC:SP:69 Specifications, the maximum gap opening is limited to 80 mm, thus up to 80 mm total movement requirement, single strip seal expansion joint may be used. For

requirement of movement in excess of 80 mm, multi-module large movement Expansion Joints like Modular Strip Seal type shall be required.

For application in Cable Stayed Bridges, it is desirable that the expansion joints possess flexibility to accommodate structural movements and differential deformation or rotations about all axes X, Y and Z. In other words, the expansion joints shall possess all six degrees of freedom.

The traffic volume on any important cable stayed bridge is significant and may easily range between 100000 - 200000 vehicles per day. Therefore, the joint design must especially account for fatigue and wearing design.

11.3.1 *Single Strip Seal Expansion Joint*

Single strip seal expansion joints consist of two heavy duty steel edge beams with anchorages in the form of Shear Headed Studs or Plate with Loop anchorages and an elastomeric sealing element specially shaped to lock mechanically inside the cavity of steel edge Beam. This mechanical locking of the elastomer seal not only guarantees the water tightness of the joint, it also ensures that the system integrity is maintained even at extreme opening and at differential movement as well as rotation condition in the structure. **Fig.11.17** shows a typical Single Strip Seal Expansion Joint.

The maximum gap width during opening of joint between the edge beams at road surface shall be limited to 80 mm for safe passage of' the traffic. The heavy-duty edge beams protect the adjacent bridge deck concrete from damage due to vehicular impact and also transfer the vehicular load to the deck structure through robust anchorage system.

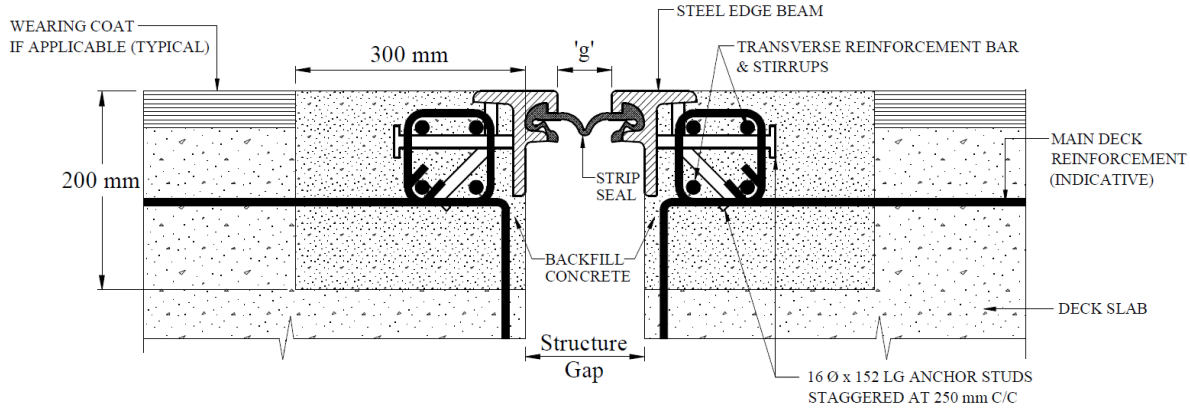


Fig. 11.17 Typical Single Strip Seal Expansion Joint

11.3.2 *Modular Strip Seal Expansion Joint*

The Modular strip seal expansion joint divides the total movement capacity into several smaller gaps. Centre beams orientated transverse to the traffic direction form the load-carrying elements and are supported by transverse support beams. Centre beams and transverse support beams are connected by assembly of steel stirrups holding the elastomeric support elements transferring the traffic-load into the bridge structure.

Geometrical or elastic gap control systems ensure equal opening & closing of all gaps under different movement conditions. **Fig. 11.18** shows a typical Modular Strip Seal Expansion Joint. In case of high seismic movements, these expansion joints may be replaced by the ones indicated in **11.3.3**.

The joint is made watertight through elastomeric strip seals which are mechanically locked inside the cavity of Steel edge and Centre beams. The modular Joint system especially for application in flexible structures like Cable Stayed and Suspension Bridges shall be able to accommodate free movement and rotation in all three axis directions.

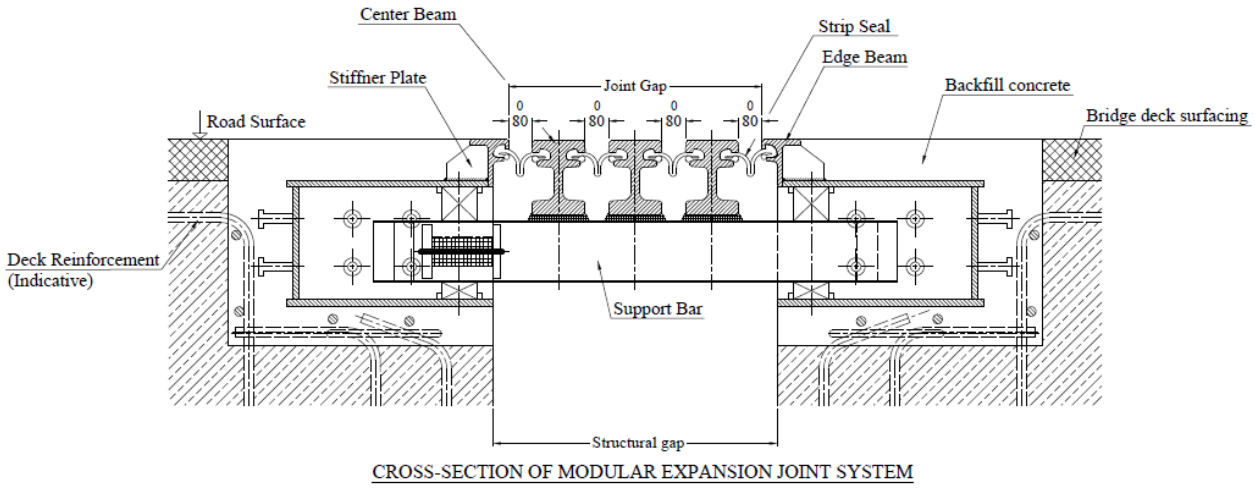


Fig. 11.18 Typical Modular Strip Seal Expansion Joint

In order for the Modular Joints to accommodate the multi-direction movement in a Cable Stayed Bridge, the Modular Joint systems having three-dimensional movement capability with flexible connection between the Centre Beams and Support Bars shall be preferred.

11.3.3 *Modular Joints to cater for large Seismic movements*

Seismic Joints either accommodate seismic displacement by means of modular lamella systems or by a special Fuse Box System (predetermined breaking point device). With the help of these two systems, damage to the Structure can be prevented or limited. Roadworthiness and crossing are ensured during and after the earthquake.

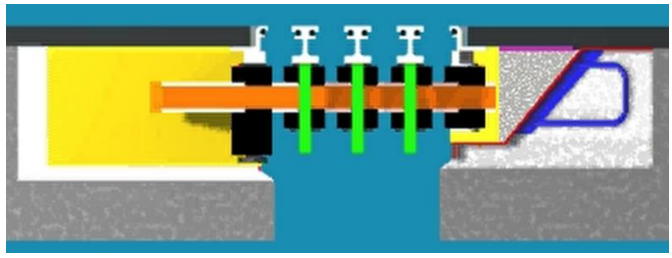
11.3.3.1 Modular Joints with Seismic Fuse Box

Modular Joints equipped with Seismic Fuse Box System (**Fig. 11.19)** allow for a more cost-efficient design of the expansion joint; it does not actually avoid but limit the possible damage at the Expansion Joint location. Fuse Box Systems permit constructive yielding of the bridge closing movements up to the anticipated threshold state (ULS) and possibly even beyond it. Depending on the Fuse Box System, the resulting closing movement is absorbed, either through an oblique-angled, upwards-deflecting plane of disengagement (fuse connection) or through a vertical lowering of the construction with subsequent sliding in the joist box. The Fuse Box System protects the deck of the bridge from excessive loads

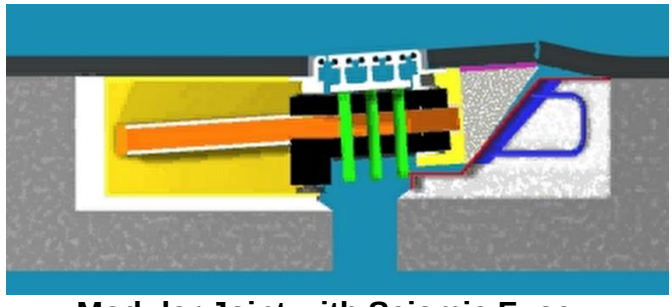
and destruction. Emergency crossing of rescue services over the expansion joint after the earthquake, once seismic fuse is activated, may be possible with caution and temporary measures. Repair of the Seismic Fuse construction and its adjacent road surface will be required, though.



Seismic Fuse Modular Joint in Production



Modular Joint with Seismic Fuse in normal working condition



Modular Joint with Seismic Fuse Activated after an Earthquake

Fig. 11.19 Modular Joint with Seismic Fuse Box

11.3.3.2 Modular Joints with large Seismic movement capability

The Seismic movement modular Joint (**Fig. 11.20)** is an enhancement of the earlier Modular Strip Seal Expansion Joint, adding, considerably, to the range of application where large and complex movements are anticipated caused by the reasons associated with geometry, movements due to earthquake, , or applications in restricted space, the use of conventional Joints is not suitable. This type of joints are able to take large seismic movements and remain fully functional and shall not require any restoration after the seismic activity.

This type of Joint is suitable for variable deformations along the bridge structure. The joint not only follows the main movement of the bridge in carriage-way direction but also distinctive movements in the 2 spatial directions perpendicular to the main direction. Even rotations of the bridge about the three spatial axes are easily borne, thereby making these Joints virtually limitless to cope with any direction and magnitude of movement and rotation.

The centre beams of the Seismic Modular Expansion Joint can slide on obliquely arranged swiveling support bars with the help of sliding bearings. Lift-off from the sliding bearing is prevented by means of a prestressed sliding spring that is arranged in the support stirrup underneath. Stirrups provide constant prestressing that is geometrically controlled.

By means of this geometrical arrangement of the support bars, the position of the centre beams is controlled such that the overall width of the joint opening is equally subdivided into the joint gaps between the centre beams and between the centre beams and edge beams, respectively. This simple and effective control mechanism results in unrestrained absorption of movements without any defined direction and simultaneous transmission of traffic loads safe guarding additional control elements.

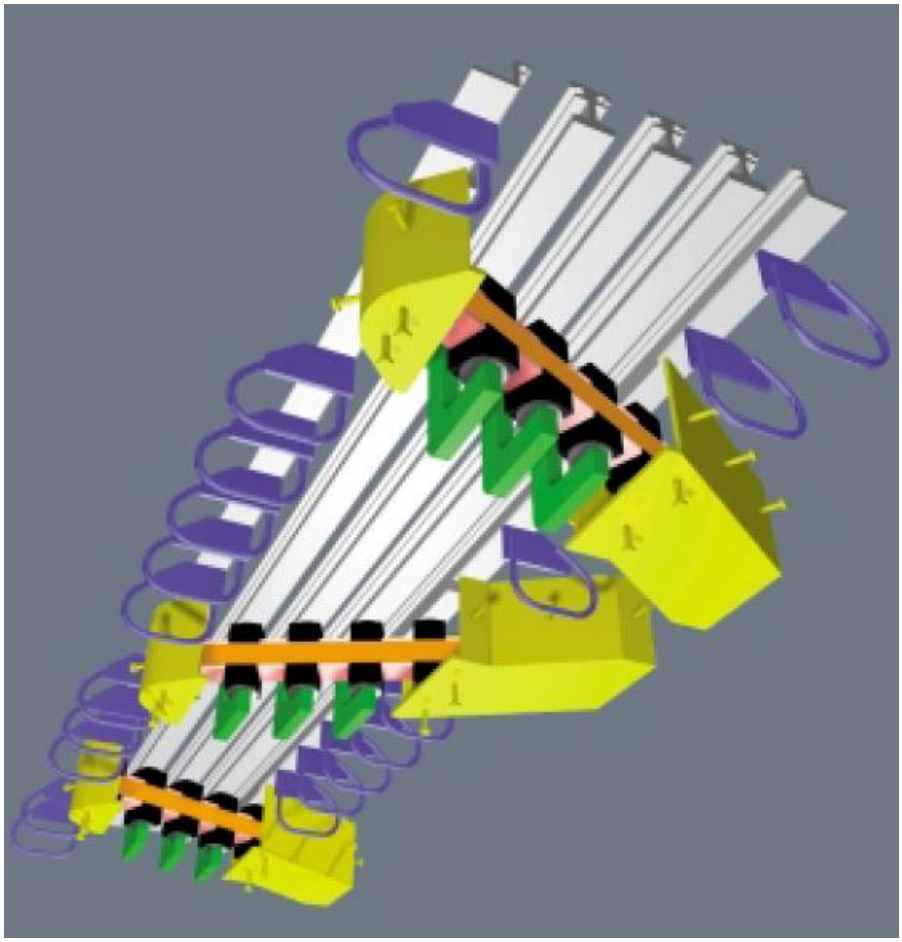


Fig. 11.20 Typical Large Seismic Movement Modular Joint construction

11.3.4 *Detailing of Expansion Joint in Crash Barrier*

At Crash Barrier Locations where Joint Gaps are small say 80-100 mm, the Expansion Joint may be provided with a simple oblique upturn so as to prevent the rain water falling underneath from the Joint end. However, for large gap opening especially in cases were lateral movements are also anticipated, leaving such large openings at the Crash Barrier location uncovered could lead to safety concerns.

Hence for large Gap opening at Crash Barrier where no significant lateral movement occurs, the gap shall be either covered with a metal cover plate system having freedom to slide and bridge the variation in Gap due to thermal movements.

Alternatively, the modular Expansion Joint may be designed to follow the Crash Barrier profile (**Fig. 11.21)** and installed accordingly to run up to the full height of the Crash Barrier.

For large to very large Gap openings especially where significant lateral movement is also anticipated, for instance in Cable Stayed Bridges, it is also a practice to provide dummy Steel construction on top of the Modular Expansion Joint Centre Beams that follows the Crash Barrier Geometry so as to maintain the aesthetics while covering the Gap opening.

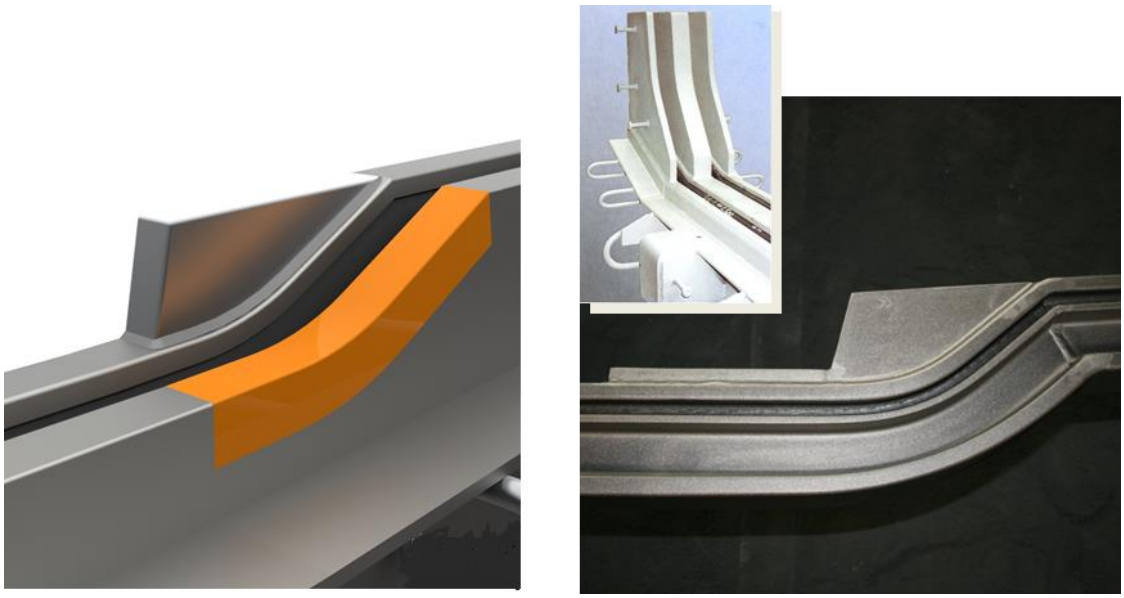


Fig. 11.21 Detailing of Expansion Joint at Crash Barrier Location

11.4 Seismic Protection Devices

11.4.1 Seismic Isolators

When the Bearings, in addition to satisfying the basic feature of transferring Vertical Load, are also required to fulfill the task of Isolation and energy dissipation under Transient Seismic events, special Isolation Bearings may be utilized for the purpose.

There are 4 main functional requirements of Seismic Isolators:

i.) Transmission of vertical loads

ii.) Lateral flexibility

iii.) Re-centering capability

iv.) Energy dissipation by Damping

Isolation Bearings are basically of two types:

i.) Elastomeric Isolators

ii.) Mechanical Isolators: Sliding Isolation Pendulum Bearings (SIPS)

11.4.1.1 Elastomeric Isolators

Elastomeric Isolators finds their field of application both in Buildings as well as Bridges. They offer economical solution, additional damping is often necessary as the damping of the elastomeric bearings, even with high damping elastomer bearings is quite low. Although they do not suffer damage in case of design earthquake, sometimes have problems with fire resistance.

Elastomeric Isolator Bearings (Usually Lead Rubber bearing wherein low damping elastomer is used in combination with Lead Core) is often found a reasonable and effective solution to be used in small span cable stayed bridges as support cum isolation system, thereby mitigating the risk of damages due to earthquake and wind forces. **Fig. 11.22** shows some typical Rubber Isolators.

Elastomeric Isolators with lead core increases its damping properties. While the lead core offers high damping by way of plastic deformation, the re-centering is still achieved by elastomer itself. The stiffening effect of Lead Core against the Elastomeric restoring capacity is to be considered. Purity of Lead Core (99.9%) is extremely important.



Fig. 11.22 Typical Types of Rubber Isolators

Elastomeric Isolators are more suitable for application where the Seismic effective periods are comparatively small and also the maximum seismic displacements and peak Load values are less.

11.4.1.2 Mechanical Isolators (SIPS)

Mechanical Isolators (**Fig. 11.23)** in the form of curved surface slider or pendulum Bearings rely on sliding friction and pendulum principle, respectively, to offer the desired load transfer, isolation, energy dissipation and re-centering behavior. A reference may be made to IRC:SP:114 for the same.

While the SIPS can be used both for non-seismic as well as seismic isolation, they can offer elongation of effective period far beyond the range of Elastomeric Isolators which can

be combined with large rotation and displacements of say +/- 1000 mm or more. Since, no separation at the sliding surfaces occurs, no significant wearing takes place,

In SIPS unlike the Elastomeric Isolators, the time period is totally independent of the Load condititon and can be precisely defined by selecting the required radius of the curved surface. Re-centring capacity and energy dissipation / sliding behaviour also does not depend on load, ageing or even temperature as in the case of Elastomeric Isolators.

SIPS can be designed for extremely high loads, yet offering smaller footprints by adopting the telescopic double curved slider geometry with no spring effects as compared to Elastomeric Isolators. Direct steel-steel-contact can also be avoided by utilizing superior sliding materials as per the relevant bearing codes resulting is service life time of 50 years and longer.

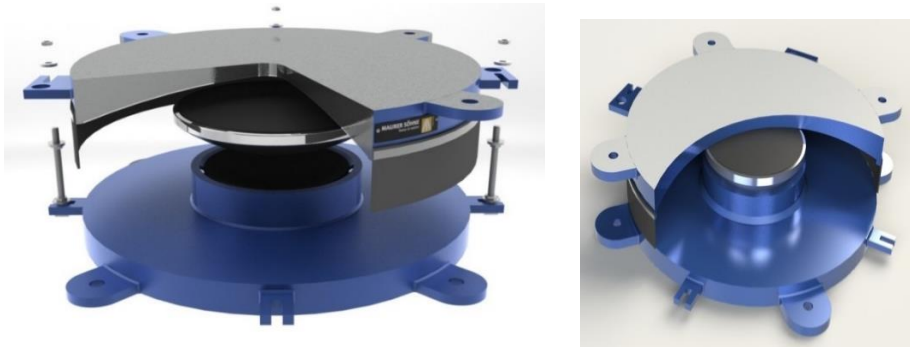


Fig. 11.23 Mechanical Isolators

11.4.2 *Dampers & STUs*

11.4.2.1 Hydraulic Dampers

Hydraulic Dampers (**Fig. 11.24)** are velocity dependent devices. They allow slow movements typically (< 0.1 mm/s) without appreciable resistance say (–< 5% of design Force), but resist sudden displacement (due to service force impacts like wind, traffic or that due to Seismic) without appreciable deformations (0.5 - 3% of stroke capacity in loaded direction). When overload they are able, depending on their design, to dissipate energy for seismic load cases.

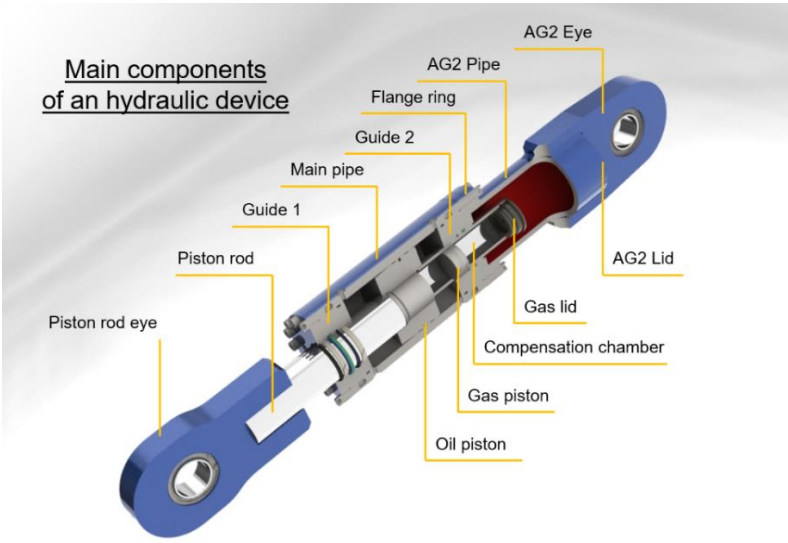


Fig. 11.24 Hydraulic Damper

Advantages of Hydraulic Devices

1) It can offer efficient energy dissipation with a variety of hysteretic loops.

2) Cost reduction resulting from reduction in seismic demand.

3) Leak free due to very low pressure during the operating period & triple sealing

4) Damping exponent available from as low as 0.04 and upwards up to 2 to suit the Structural demands.

5) Short response time due to low compressibility.

6) Behavior independent of Temperature variation.

7) Lock-up of service impacts

8) Design considers even service load cycles for fatigue

9) Accommodation of accumulated sliding path of 50 km or more possible.

Hydraulic devices, in general, can be distinguished in three main types:

i.) Shock Transmission Unit (**STU**)
ii.) Shock Transmission Units with load limiter (**STL**) and
iii.) Hydraulic Damper (**HD**).

While at low velocities the behaviour of all the three devices are the same, at high velocities the behaviours differ from each other (**Fig. 11.25)**. All devices can work in a regular range of - 40 to 70°C, which can also be adjusted on demand. General principles of working of Hydraulic Devices are enumerated in **Fig. 11.26**.

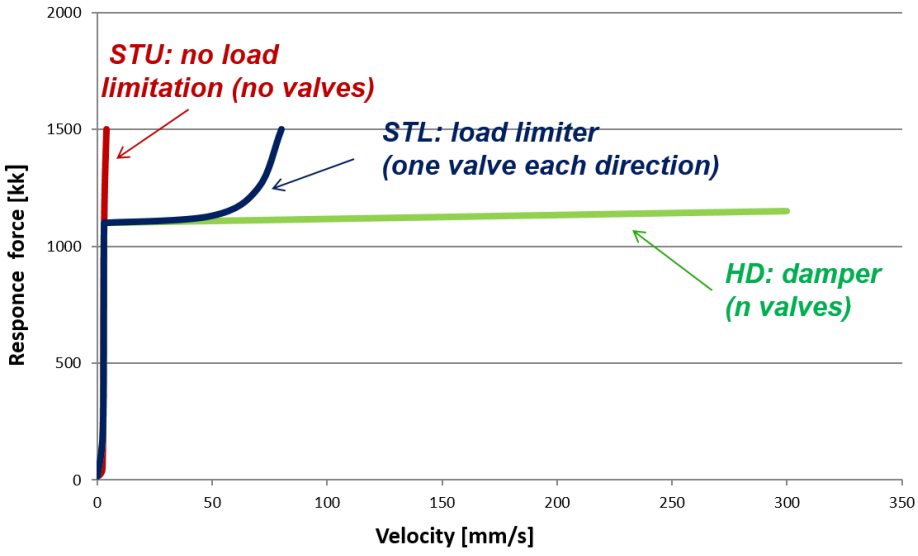


Fig. 11.25 Behavior of Various Hydraulic Devices

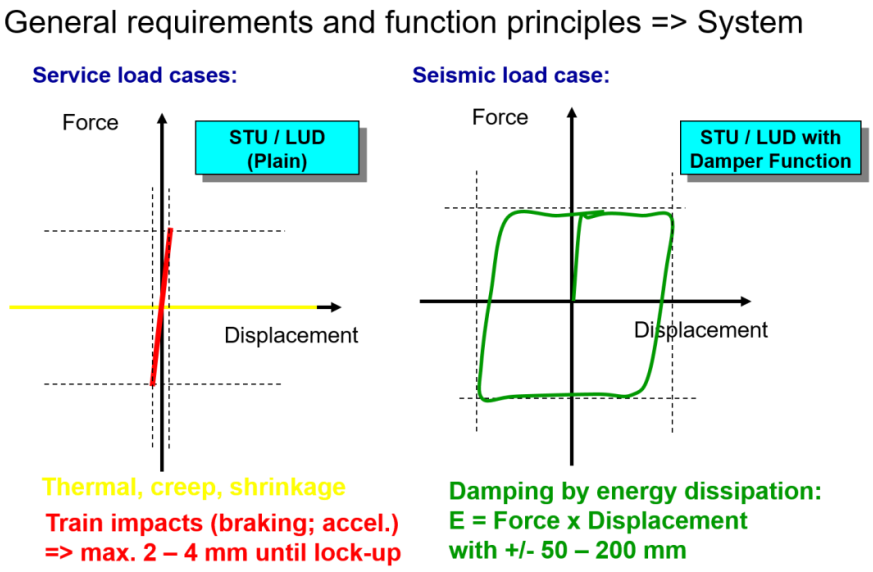


Fig. 11.26 General Principles of Hydraulic Devices

All Hydraulic devices, irrespective of their types, shall be provided with a Spherical Hinge (**Fig. 11.27)** for Pin connection so as to facilitate minor misalignment or limited lateral rotation / displacement without hampering its function. The typical value of misalignment that can be handled with this mechanism is usually limited to 4%.

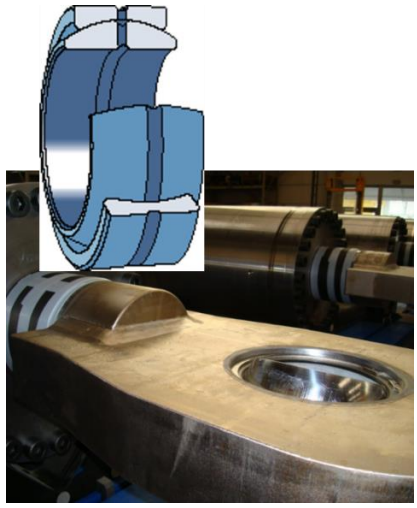


Fig. 11.27 Hydraulic Devices: Spherical Hinge

11.4.2.2 Shock Transmission Unit (STU)

Low velocities displacements of the structure caused by thermal actions, creep and shrinkage, are accommodated without appreciable resistance (maximum 10% of the nominal force) by the STUs. This behaviour aims at allowing free slow acting movements, without inducing undesired forces acting on the structure.

For higher velocities of the structure the device locks up starting typically from 1 mm/s and transmits longitudinal dynamic actions without appreciable displacements. The devices exhibit low elasticity levels, about 2-5% of the stroke. Scope of the hydraulic shock transmitters in this phase is that of creating additional temporary restrains under short-term dynamic actions (longitudinal seismic actions, strong braking or wind gust) in order to distribute the force to the supports other than the restrained pier/abutment or predetermined braking points for better distribution. **Fig. 11.28** for fixing typical arrangement

If the velocity raises beyond the design input, the response force increases up to failure of the device, its connection or the Structure itself.

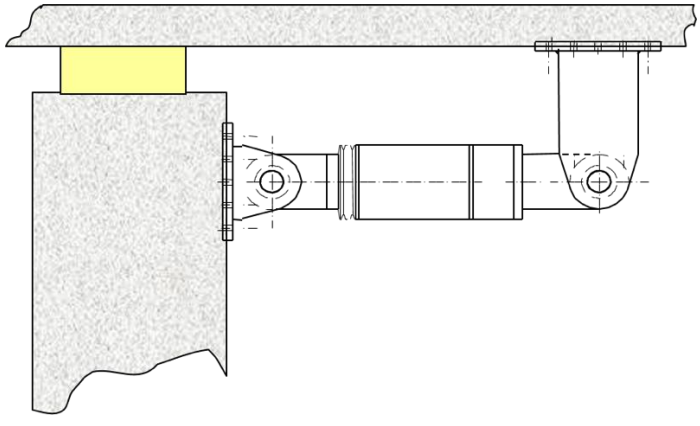


Fig. 11.28 Typical Fixing Arrangement of STU

In the shock transmission units, both resistance to the movements due to thermal variations and deformations consequent to an earthquake attack have been minimized, the velocity V_0 is usually individually adapted in the range from 0.1 to 1.5 mm/s, but for very large structures can reach the value of 5 mm/s.

11.4.2.3 Shock Transmission unit with load Limiter (STL)

Due to the fact that tolerances exist:

a) within the structure,

b) within stiffness of each device,

c) within the occurring lock-up velocity of each device, and

d) within the STU-bracket system

the parallel and serially arranged STU's will not get activated simultaneously. Instead, each one of them will develop a different response force.

In case the STU is designed without force distribution function, the difference in tolerances effect is a considerable higher input force on some devices. For this reason, specifications require STU to be designed considering 150% of the rated design force. Moreover, this increase in design force is not only applicable for the STU device but for the support brackets, the anchor system and finally for the structure itself, too. Yet, this increase in input force for design may or may not be even sufficient and the overall system is still not fail-safe.

An STL (**Fig. 11.29)** on the other hand displays the same working principle as an STU, but offers the possibility of a load limitation. One valve in each direction opens after reaching a predetermined pressure level, the device yields causing a movement of the structure. Thus, the transmitted longitudinal forces are limited to a desired value in the case unexpected seismic actions exceeds those resulting from the design level earthquake. This option aims at avoiding damages to the structure (piers or abutments), anchoring devices, and the shock transmitter itself.

STL are also used to ensure that all shock transmitter in a structure activate together.

The load is limited up to a certain velocity. For higher velocities the valve section is too small to allow a fluid flow Q = v*A (continuum equation) through the chambers, and the force begins to rise again.

The reliability factor of shock transmitters on their design force shall be $\gamma x=1.5$, unless an overload protection system or "load limiter" is incorporated. In this case, the value of the reliability factor can be reduced to $\gamma x =1.1$ and shall be applied to the design system force F_0 specified by the designer. The adoption of STL decreases the forces acting on the structural members by say 26%. It increases the overall safety of the devices and the structure as it is granted that all devices in serial and parallel arrangement are equally and simultaneously loaded when affected by sudden service or seismic impacts. This is not the

case with classic STUs. These might be overloaded even with more than the reliability factor of 1.5 applied onto the design force. Therefore the STL application reduces costs of the structural members and even cost of the shock transmitter itself, because an STL is more compact, i.e. smaller than an STU. The STL is an economical solution, while providing additional technical benefits and reliability.



Fig. 11.29 Shock Transmission Unit (STU)

Advantage of a Shock Transmitter with Force Limiter (STL)

i) STUs without overload protection require a safety factor of 1.5 (pr EN15129, chapter 5.3.3)
ii) STUs with overload protection need only a safety factor of 1.1
iii) This means that adjacent structures can be designed for a 26 % less response force when an overload protection is incorporated into an STU.

Correspondingly, savings in the adjacent structure can be realised

11.4.2.4 Hysteretic Dampers

The Hysteretic Dampers (**Fig. 11.30)** unlike the Hydraulic Devices are displacement dependent devices. These safety devices are more often used in Building Construction but also finds their application in bridge constructions. Usually, Steel-Hysteresis-Dampers are installed in combination with re-centering elements, e.g. elastic columns, elastomeric bearings or sliding isolation pendulum bearings etc.

Hysteretic Dampers exhibits the following Advantages:

i) Higher initial stiffness for small displacements

ii) Re-centering might be not always sufficient and might be enforced by additional devices

iii) Max. force at max. displacement

iv) More effective displacement control possible, compared to bearing/isolator system alone.

v) Highly efficient energy dissipation

vi) Temperature independent

vii) Aging- resistant

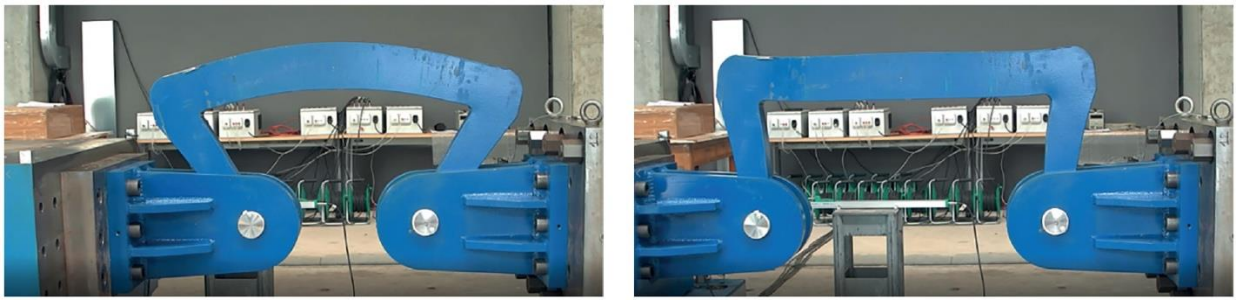


Fig. 11.30 Hysteretic Damper

11.4.2.5 Tuned Mass Dampers (TMD) (**Fig. 11.31**)

Cable Stayed Bridges are susceptible to issues relating to vibrations due to their low damping in combination with rather low natural frequencies. Vibration and vibration displacements are required to be damped which otherwise would render the structure non-usable under peak excitation or cause unacceptable discomfort. Also, resonance phenomenon can occur and structural collapse could happen. Fatigue phenomenon with crack in the structure can occur, which can lead finally to the structural collapse, too.

Tuned Mass Dampers are deployed to dampen the vibrations and uncontrolled displacements in the structure that are induced due to excitation forces from Wind, Seismic, etc.

As the name implies, a mass equivalent to the kinetic mass of the Structural component is suspended using springs and damping mechanism to act against the free structure swing or displacement thus controlling its movements.

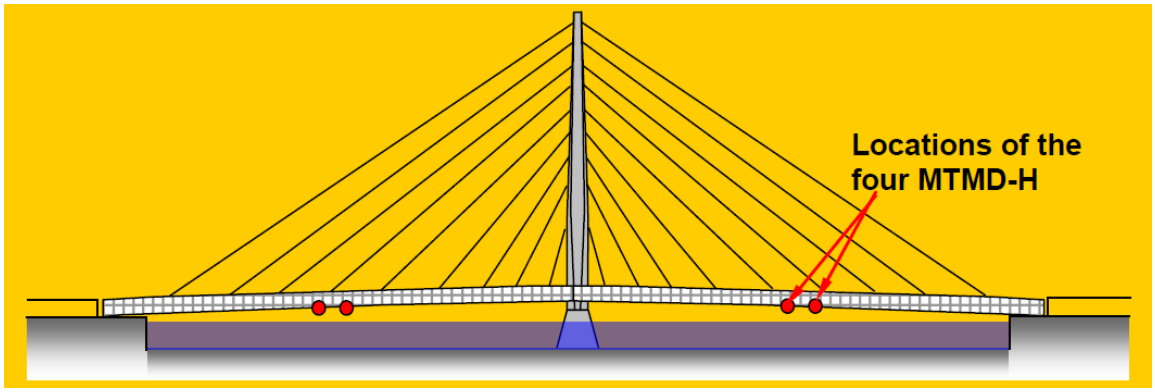
TMDs offer following benefits:

i) Acceleration reduction for wind

ii) Low eigendamping → long decay durations

iii) Effective reduction of structural decay duration after the earthquake

A TMD is connected to the structure at the location where a significant or the biggest vibration is expected to occur. The TMD-device consists of a moving mass, springs and a damping element. The below sketches describe the principle of TMD and its main components. The TMD should be placed at the location of the greatest vibration amplitude in which case the efficiency is the highest with least effort.



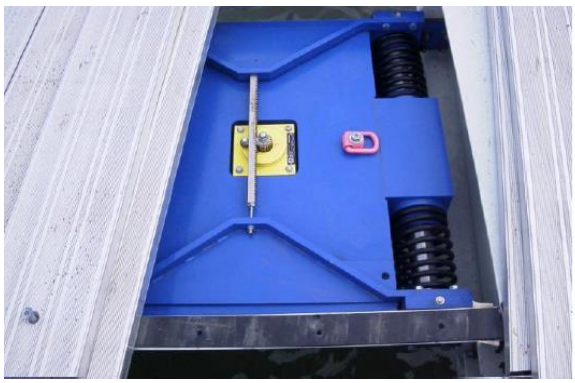




Fig. 11.31 Placement of Tuned Mass Damper (TMD)

11.4.2.6 Cable Dampers

Cables in a Cable Stay Bridge can vibrate in different modes due to varying excitation conditions:

To control multi-mode Cable Vibrations, passive, semi-active or real time adaptive Cable Dampers as discussed in detail under **Chapter 10** shall be used to dampen the varying vibration frequencies and to keep the Cables and Structure stable, saving from fatigue effects thus ensuring longer service life.

CHAPTER - 12

GEOMETRY CONTROL

Geometry control is an essential part of the construction monitoring process: appropriate profile and alignment are required as for any other bridge; in addition, geometry control is an essential tool to ensure that the stresses in the structure and stay cable forces meet design expectations.

12.1 Casting Curves

In all segmentally constructed bridges, one needs to cast the segments with camber corrections so that once all segments get erected /cast at site, one can achieve desired profile of bridge in its final shape.

Determination of pre-determined profile of segments such that the adjacent segments match each other, requires a detailed pre-camber control analysis, wherein all construction stages of erection of such prefabricated segments to build a complete bridge are modelled with accurate loading (temporary & permanent) activated at each construction stage (step) with most accurate predicted time line.

Time tag for each activity is very important so that time dependent effects of material used such as creep of concrete, shrinkage of concrete, relaxation effect of prestressing steel can be accurately accounted for. Such effects can cause deformation of each erected segment depending on stress level history in each segment during construction of bridge, age of concrete of segments at the time of its erection in bridge unit and age of concrete of segment at a construction stage at which deformations are required.

Each construction step analysis requires calculations of

a) Current step displacement ($\boldsymbol{\delta}_{ab}$) at each node:- It is displacement of nodes of selected construction stages due to activities performed within each construction stage (step)

b) Virtual Displacement at each node:- which occurs at a node before the corresponding node is even activated.

c) Accumulative / Net displacement nodes:- It is the net displacement of nodes up to end of the selected construction stage, i.e it is the summation of current displacement of node which occurs after the node is activated up to selected construction stage.

d) Real displacement of node:- It is the summation of virtual displacements which occurs before the corresponding node is even activated and net displacement, which occurs after the node is activated.

12.1.1 *Camber Control For Precast Segmentally Constructed Units*

A simple example of cantilever erection of three precast segments in three stages is given in **Fig. 12.1** Current step displacement and virtual displacement at each node at construction stages are shown. Such displacement values results are extracted by

performing stage analysis for each stage separately under corresponding stage loading. Real displacement profile at the end of erection of each construction step is shown in last figure of **Fig.12.1**.

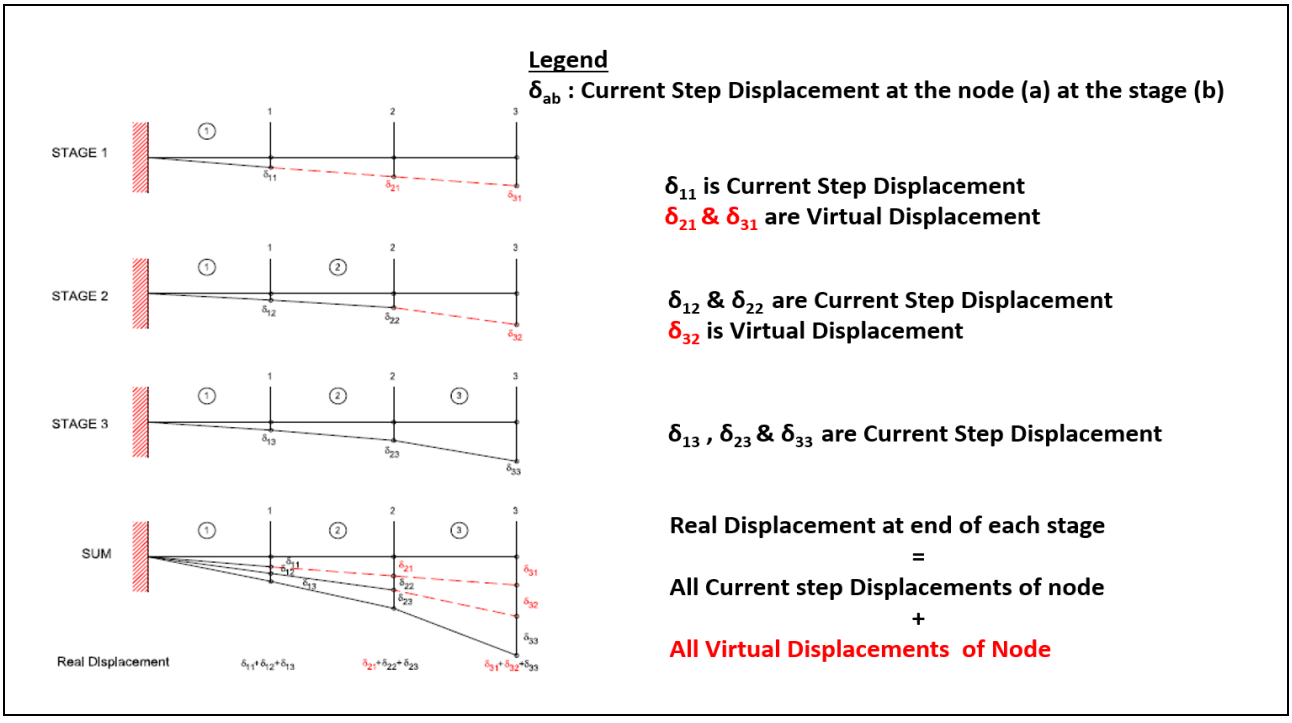


Fig. 12.1 Real Displacement at Node (For Precast segments)

Profile of erected unit at end of each construction stage with, (i) no camber correction is done & (ii) camber correction done at each construction stage, is shown in **Fig. 12.2**

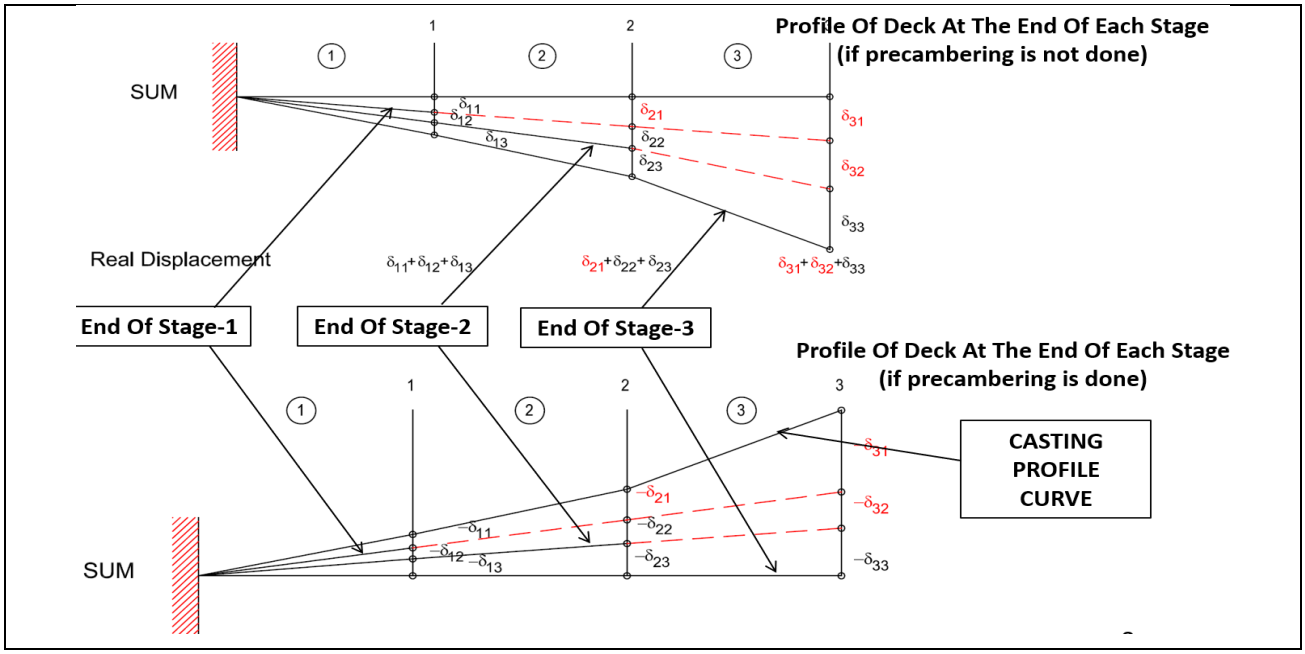


Fig. 12.2 Precambering Using Real Displacement for Precast Segments

By reversing the final real displacement profile of erected unit at end of construction, one can work out casting profile curve of segments. The angle (ψ_i) between two segments to achieve a final structure with no deformation can be worked from real displacement data **Fig.12.3**. Such calculated camber correction of angle (ψ_i) between match cast segment is required to be provided while casting of segment.

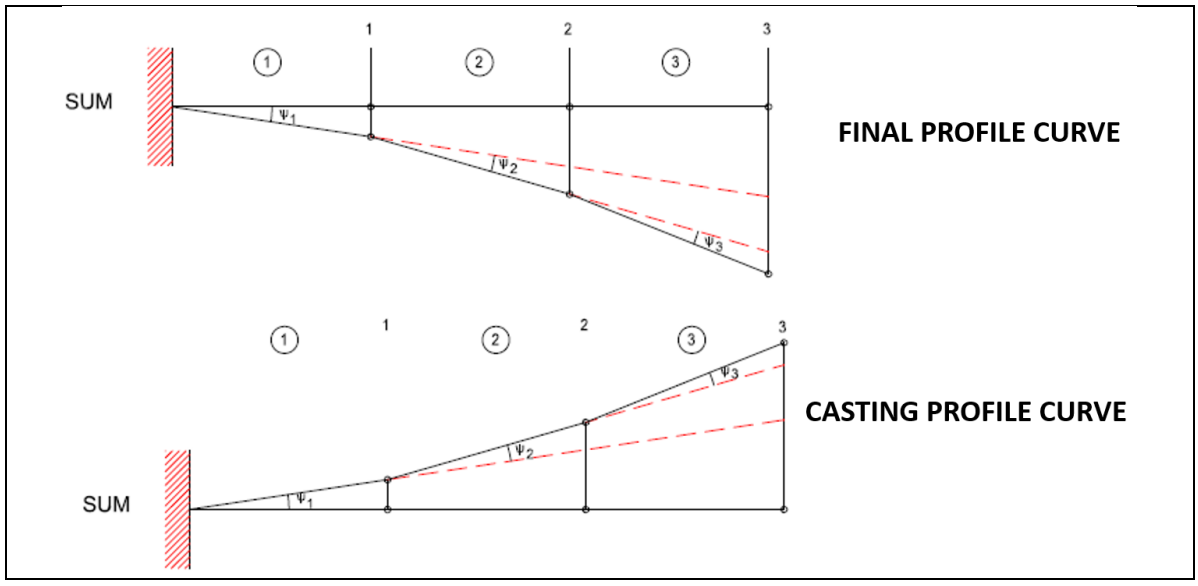


Fig. 12.3 Casting curve Using Real Displacement - For Precast segments

12.1.2 *Camber Control For Cast-In-Situ Segmentally Constructed Units*

A simple example of cantilever erection of three cast-in-situ segments in three stages is given in **Fig 12.4**. Current step displacement at each node at end of each construction stage are shown. Such displacement values results are extracted by performing stage analysis for each stage separately under corresponding stage loading.

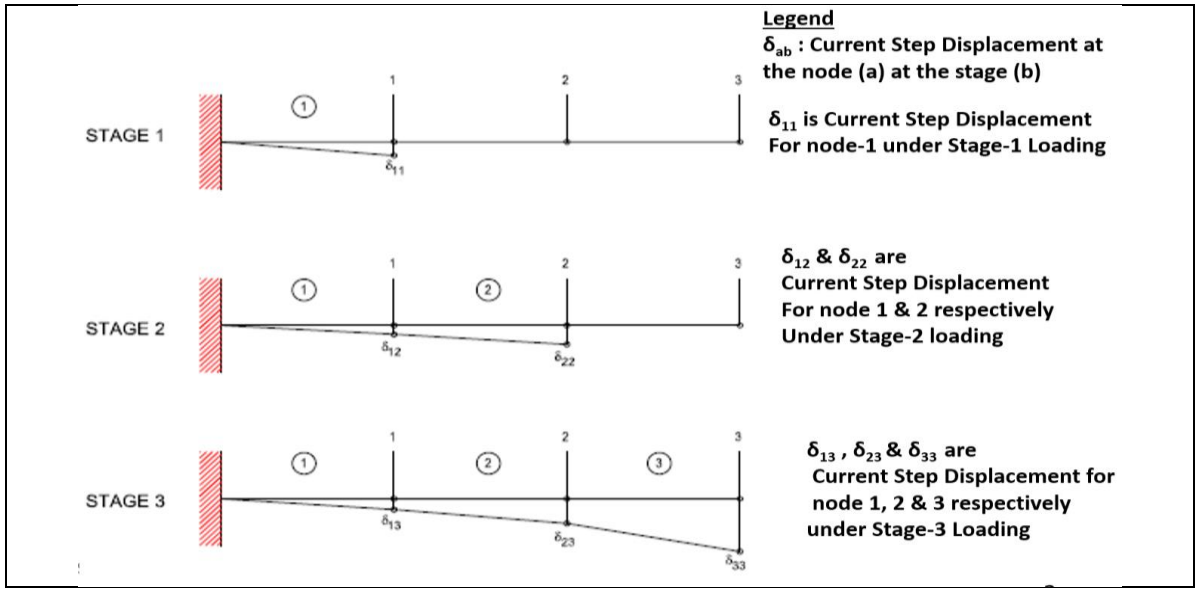


Fig. 12.4 Net/Accumulated Displacement at Node (For Cast-in-Situ Segments)

Net displacement profile at the end of erection of each construction step is shown in **Fig.12.5** (top figure) if no precamber is done.

By reversing the final net displacement profile of erected unit at end of construction, one can work out precamber value which needs to be provided at each newly casted node in each construction stage w.r.t final proposed profile curve of segments as shown in **Fig 12.5** (lower figure).

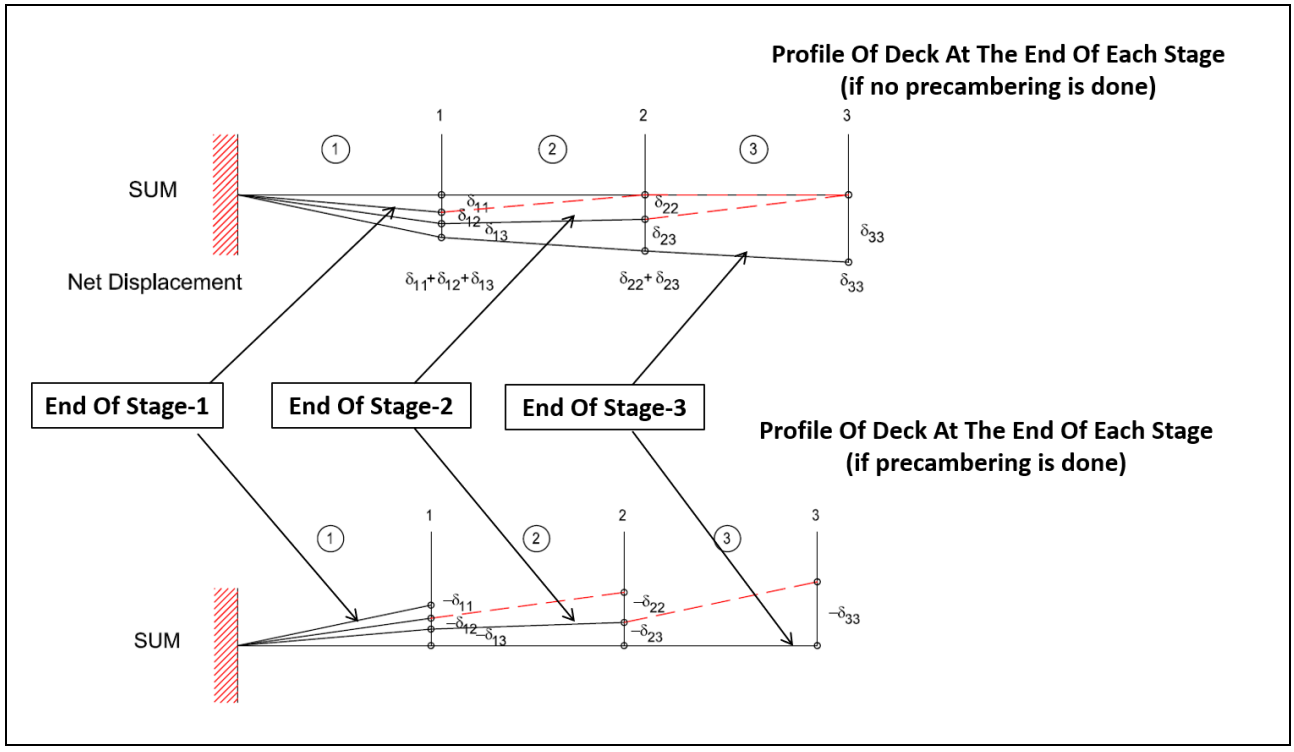


Fig. 12.5 Precambering Using Net Displacement For Cast-In-Situ Segments

12.2 Geometry Control for Precast Box Girder Segments

Segments are usually match-cast by the short cell method. Each segment is equipped with 4 survey markers (recommended at top of webs) for elevation control and 2 for alignment control (recommended at center line). The location of these markers is recorded in the casting cell reference system. “As-cast” plots are developed in this reference system.

The as-cast set of data is then transformed into the reference system of the erection site by a simple matrix operation.

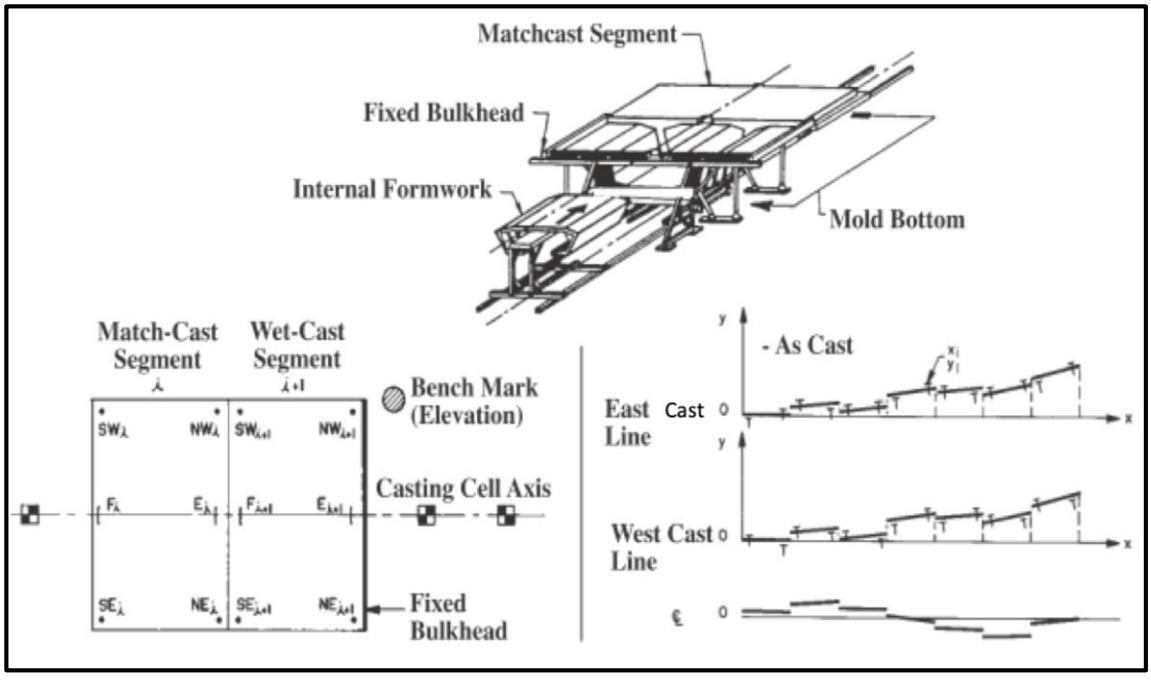


Fig. 12.6 Short Line Casting Method

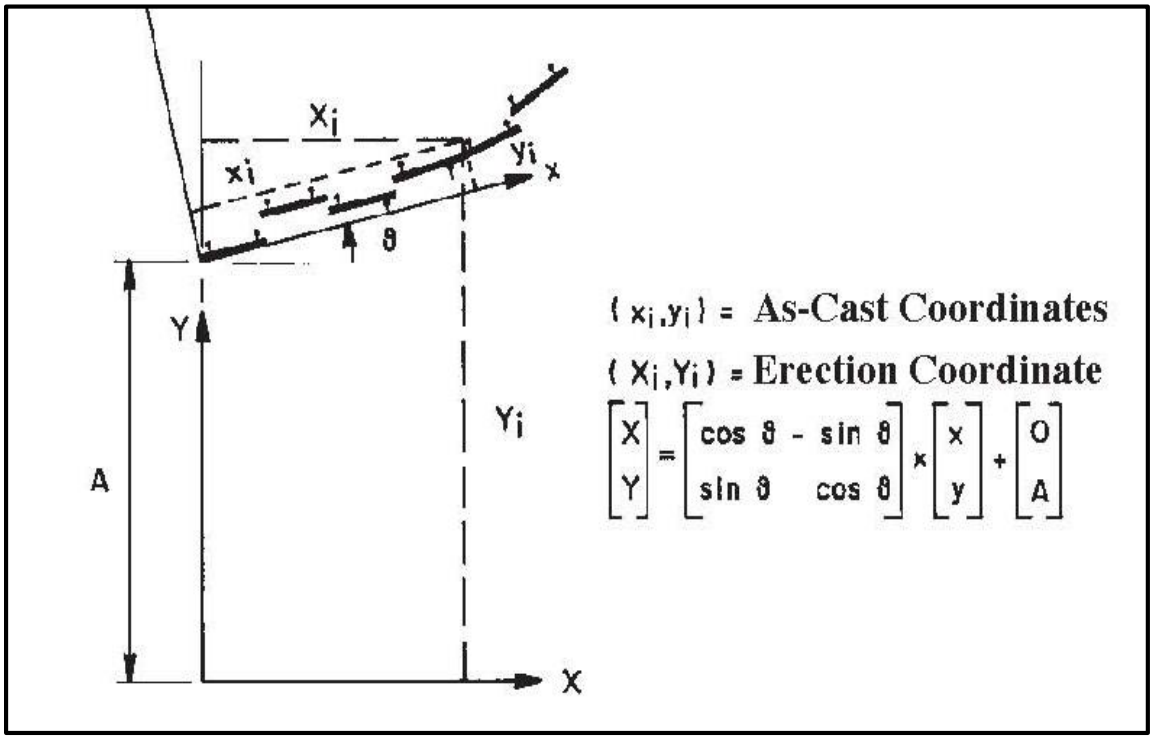


Fig. 12.7 Coordinate Transformation

It should be noted that a systematic error of only 0.3 mm on elevation measurements in the casting cell would result in significant discrepancies at the end of a long cantilever. The

first segment of the cantilever must be placed accurately to avoid offsets at the cantilever end.

Accuracy of controlling the horizontal alignment is more critical parameter especially when wider precast is planned and constructed. Wider deck such as more than 20 m width is commonly adopted in India. Misalignment of horizontal alignment result in more significant problem in closure joint stage as it's rectification in horizontal discrepancy is extremely difficult either by using the shimming plate or forced adjustment.

Deflections of the structure at each stage are obtained from the general time-dependent computer program. Adding these values to the as-cast survey marker elevations provides expected elevations at each stage of erection.

The elevation of the cantilever end should be plotted at each phase against theoretical **(Fig. 12.8)** Instant movements of deck and pylon should be measured when lifting a segment or stressing a stay cable and checked against theoretical figures. Interpretation of the results would give indications on the actual stiffness of the structure.

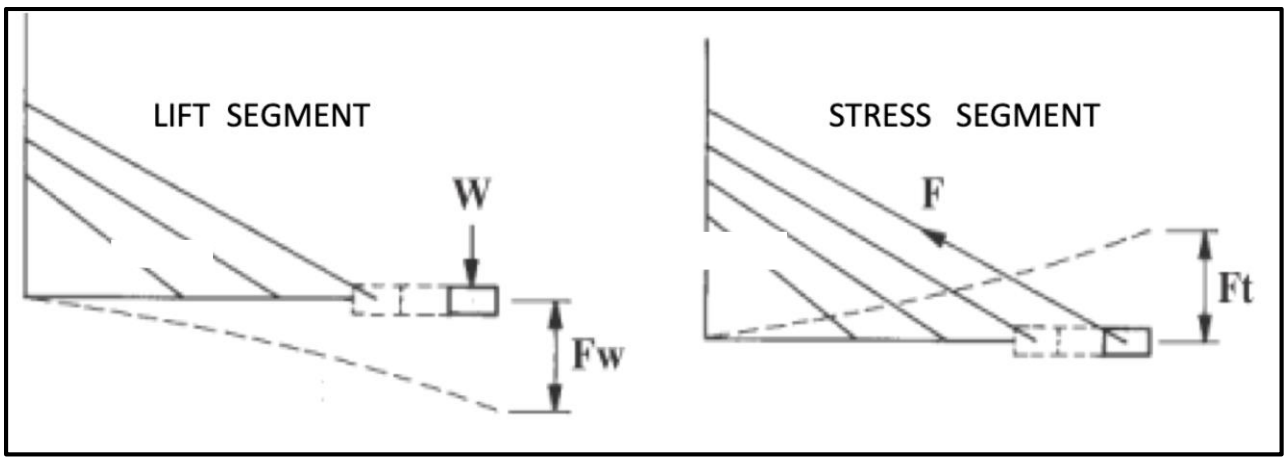


Fig. 12.8 Deformations During Construction

Bending of the overall cantilever can be checked by plotting actual and theoretical elevations at each joint at a given erection phase. After each deck survey, the verticality of the pylon should be checked. If the pylon is out of position, it means that the structure is subjected to unbalanced loads; and the deck survey would be affected by this condition.

It is imperative to check that the last erected segment is not warped. This can simply be done by comparing readings of the 4 elevation survey markers after erection and in the casting cell **(Fig. 12.9)**.

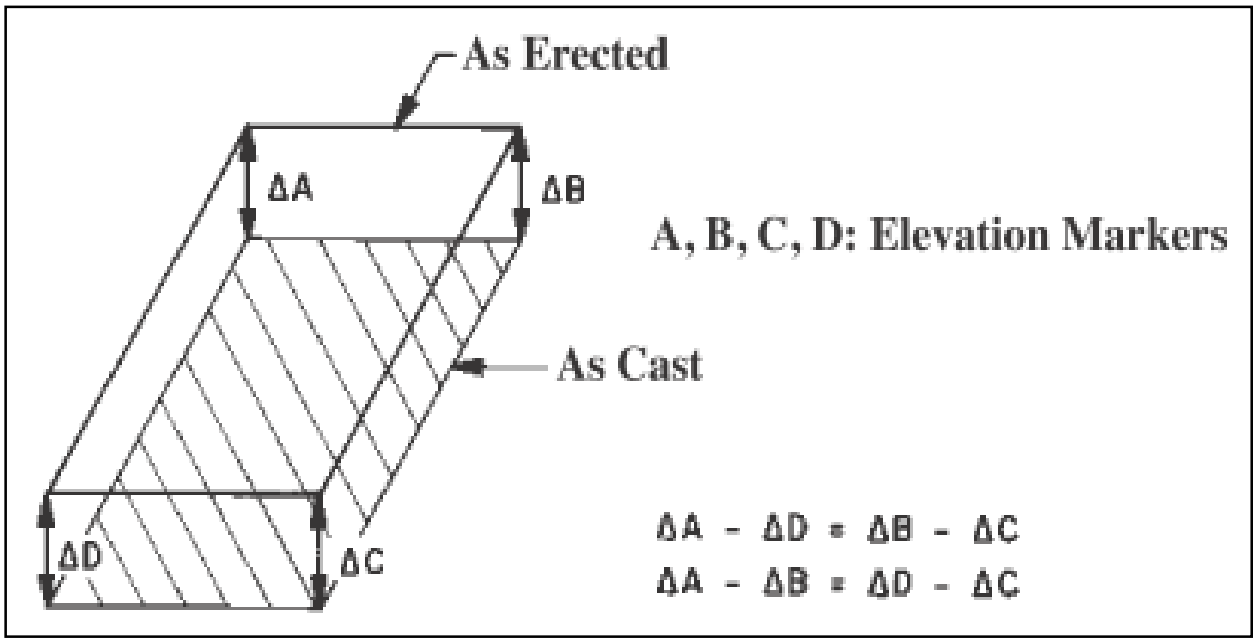


Fig. 12.9 Segment Shape Control

The accuracy of the stay cable jacking force can also be checked by measuring variations of elevations of the same survey marker during a full erection cycle (segments lifting, stay cable stressing and re-stressing). Comparing the result with theoretical figures allows determination of a gain or loss of elevation for the full cycle. Loss of elevation could indicate a deficit of effective stay cable force or excessive downward deformation, for instance, because of additional segment weight. Since the total deck movement includes an equal number of upward and downward deflections, the influence of the actual structure stiffness on the results is reduced.

The thermal gradients within the superstructure and between concrete deck and stay cables can affect stay cable forces. The nominal stay cable forces given in the design normally do not account for thermal effects. One solution consists of verifying and adjusting the stay cable forces early in the morning if the stay cable has to be stressed during the day-time in order to satisfy the contractor's operations. It is more practical to estimate the gradients at the time of stressing, for instance, with thermo-couples. At each stage of erection, a correction of stay cable force can be computed to account for loss due to thermal gradients. During the daytime, thermal gradients create a downward deflection of the cantilever. When the gradient dissipates, the cantilever deflects back up and it results into a loss of stay cable force. Therefore, when stressing during daytime, an increase of stay cable jacking forces is required.

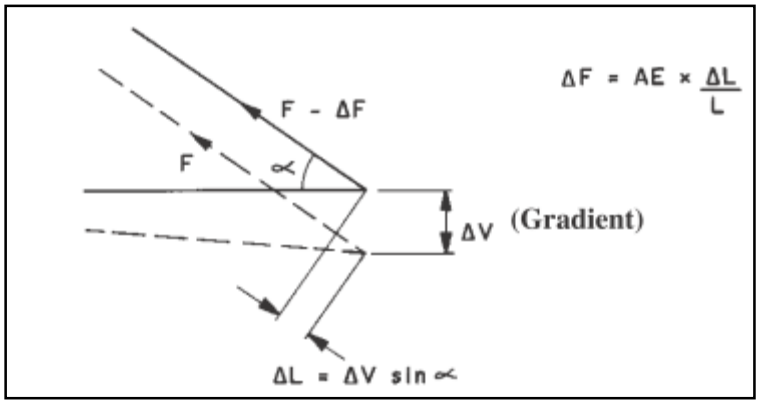


Fig. 12.10 Thermal Effect Stay Force Correction

Plotting of the cantilever end elevations **(Fig. 12.11)** at each phase gives a good indication of the actual stiffness of the structure.

The deck deflects up and down under stay cable stressing and segment lifting. The amplitude of the deflections increases with the length of the cantilever.

If the bridge is stiffer than assumed, the cantilever tip elevation will seem high after lifting a segment. After stressing a stay cable, the elevation of the cantilever will seem low.

Modifying the stay cable forces to bring the deck to the theoretical elevation at each phase could result into unacceptable stresses in the superstructure. This explains why a cable stayed bridge with a rigid type superstructure cannot be erected following theoretical elevations only. In this case the erection should be governed by stay cable forces with a tolerance not exceeding +/- 5%.

If cantilever tip elevations are low after stressing and lifting a segment, this is an indication that the whole cantilever is aimed low or is deflecting excessively.

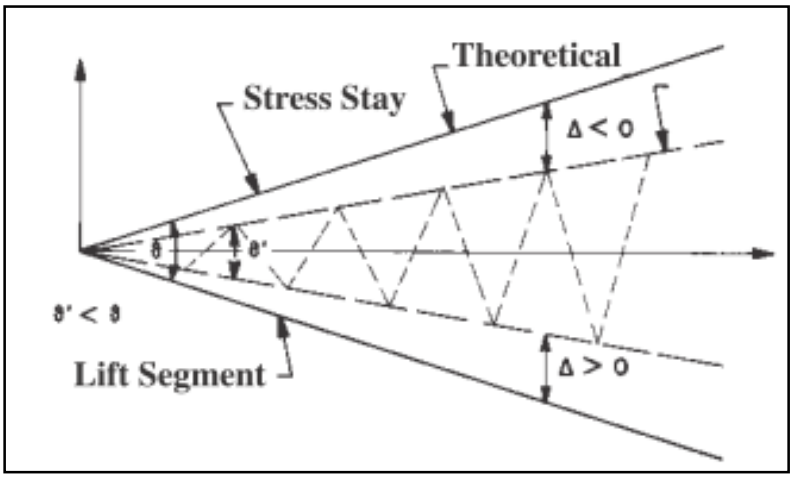


Fig. 12.11 Plot of Cantilever Tip

Shimming the joint to adjust the profile during the erection may be required similar to the precast balanced cantilever erection if the deviation of erected profile is large such that adjustment by stay cable force is insufficient to rectify. This case would happen more in case where girder is stiffer and span length comparably shorter. (i.e., stiffer deck compared with span length).

The American practice is to use the shims made of ASTM 240 type 304 wire cloth (roving). The GRP (Glass reinforced polymer) shim plates are recommended as the modulus of elasticity of hardened epoxy glue is closer to GRP avoiding potential stress concentration. In any case the maximum thickness of shims shall not exceed 6mm.

12.3 Geometry Control for Cast in Place Box Girder Segments

The geometry control procedures differ slightly from the procedures used for precast box girders: local geometry adjustments can be made with the traveling forms instead of shimming the joints between precast segments. Like with precast box girders, geometry discrepancies due to additional segment weight or inaccurate stay cable jacking forces need to be corrected by re-stressing the stay cables.

12.4 Geometry Control for Flexible Decks

Except for the initial casting stages close to the pylon, the deck elevations can vary greatly with a small variation of the stay cable force. At the same time, stresses in the deck are not affected significantly by variations of the deck geometry. It is therefore logical to prioritize the deck geometry control over stay cable jacking forces. Variations of segment weights are then automatically compensated for but it is important to record the stay cable forces to verify that allowable stay cable stresses are not exceeded. Again, it is advisable to provide some additional capacity in the stay cables at the design stage to account for these weight variations.

Since the erection procedure is guided by the deck geometry, it is essential to apply the stay cable jacking force at a time when the geometry of the structure is not affected by thermal gradients, that is early morning before sunrise.

12.5 Geometry Control for Steel-Concrete Composite Decks with Precast Concrete Slabs

Composite deck, consisting of longitudinal steel girders and floor beams onto which precast or cast-in-situ concrete is connected via shear connectors is advantageous for construction of bridge spans in the range of 400 m - 600 m.

For the construction of composite steel-concrete decks of cable-stayed bridges, methods of erection and analysis have to be applied that, upon completion of the deck, accurately yield the prescribed dead load configuration of the deck regarding geometry and forces. During deck erection, no unwanted bending moments should be locked into the composite section when the concrete slab is connected to steel girders and floor beams. The advantage of using precast concrete deck slab is to minimze the inclusion of locked-in bending and the respective effects of concrete creep & shrinkage.

The aim should be to eliminate or at least minimize the possible effects of concrete creep & shrinkage on the geometry and forces of the deck in order to simplify analysis of the deck erection, as well as to increase the accuracy of the geometric predictions. These effects can be minimized if the deck is erected so that the internal forces for the dead load configuration are those of the continuous beam on rigid supports. This is so because in a continuous beam on rigid supports, the sum of the positive and negative bending moments- that is, the span wise overall curvature- is zero. Moment variations due to creep eliminate each other (due to the axial forces in the deck, the deck slab can be assumed to be un cracked). The initial moment distribution does not change with time, and therefore time-dependent effects due to moment creep do not have to be considered. This method minimizes locked-in curvatures and the respective moments in composite decks. The typical erection cycle for a deck segment is defined by four stages and a two-step tensioning procedure for the cables as briefly explained below:

1) Erection of prefabricated welded steel grids consisting of longitudinal main girders with a cable anchorage at their tips and cross girders by the cranes and bolted to the deck.

2) Installation and stressing of stay cables.

3) Erection of concrete decking panels and placement of in-situ concrete in closure joints between panels.

4) Re-stress the stays cables once the closure joint concrete has attained the required strength.

12.6 Geometry Control of Pylon

The geometry of the pylon **(Fig. 12.12)** must be controlled during construction to verify that the maximum out-of-plumb and out-of-straight-line values assumed in the design for this compression member are not exceeded.

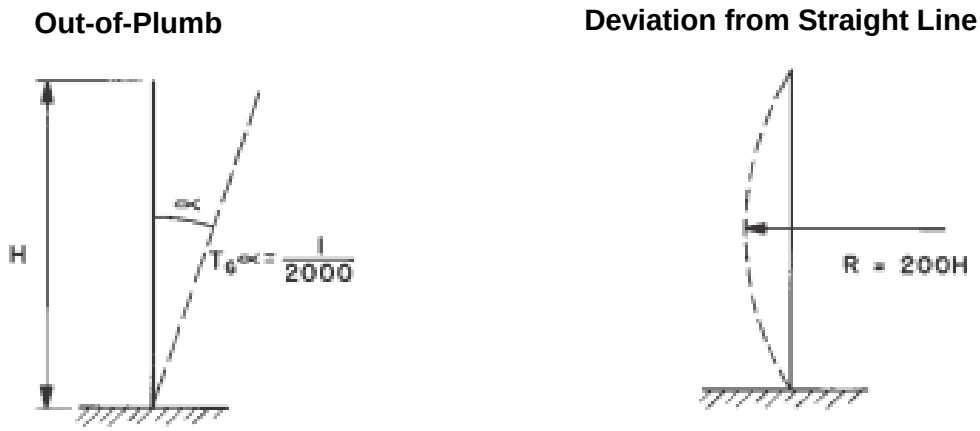


Fig 12.12: Pylon-Construction Tolerances

12.6.1 *Geometry Control of Steel Pylon*

12.6.1.1 Geometric Control during Fabrication

Invariably steel pylons are constructed by prefabricated segmental construction method where segments are fabricated in an established fabrication workshop and then transported over to site, while they would be reassembled before erection.

Some of the major challenges involved in fabrication of steel pylon are :

- Use of plate having thickness ranging from 25 mm even up to 250 mm including Z quality steel. Adoption of appropriate welding sequence to avoid deformation
- Preheating up to temperatures in access of 600 degree for butt welding of thicker plates
- Stress relieving arrangements due to heavy welding
- High precession and tolerance requirement for bolt holes and end milling
- Drilling millions of holes for HSFG bolts ranging from 12 to 36 mm in very thick plates.
- Availability of Skilled Manpower
- Requirement of special lifting equipment's during fabrication and trial assembly
- Availability of well-equipped fabrication facility.
- Machining facility to carry out sometimes even up to 36 m long surface at one go.

Adopting the most appropriate welding procedure is very important to avoid distortions and deformations for welding of pylon segments, and to ensure high level of accuracy and quality. Fabrication accuracy would directly affect geometrical dimension of the whole segment after their trial assembly, which were to be precisely matched within 1/10 of mm. Number of NDT testing like Dye penetration test, ultrasonic test, etc. have to be carried to ensure the desired quality of the product.

All the compression joints are normally designed to transfer load through bearing contact surfaces along with splice connection. To achieve appropriate bearing surfaces, all the contact surfaces in compression like main girder edges, pylon horizontal joints have to be machined to have a surface planes not more than 0.3 mm per meter.

Following are some typical steps recommended in the fabrication process :

- Technical groundwork – preparing fabrication drawings, workout requirement of material, equipment's and support arrangements.
- Procuring and testing of Raw material

- Cutting of material as per approved fabrication drawings.
- Panel welding and NDT testing
- Fit up of an segment and NDT testing
- Initial survey and dimensional check for fit-up segment. Conduct rectifications if required
- Machining end surface of the segment in contact with other surface at splice, which transfers load through bearings.
- Match drilling
- Trial assembly, match drilling and final survey
- Blasting and painting.

Trial assembly at fabrication yard is an important prerequisite for prefabricated segmental erection of steel pylon similar to that match casting in concrete segmental construction. Trial assembly has to be performed in line with the sequence of erection at the fabrication factory / bridge site. The trial assembly jig has to have enough rigidity to avoid subsidence.

The special attention regarding the following key points are warranted to decide the fabrication, trial assembly and geometric control procedures :

- The accurate geometric dimension of pylon structure with edge considering their complex configuration if any with heavy welding in volume, the control in welding shrinkage and in the structure deformation during and after welding.
- The quality of thick-plate welding.
- The requirement on the accuracy of bearing contact in end face considering their huge vertical compression of the pylon tower and its shaped alignment.
- The trial assembly has to be carried out to ensure the geometrical alignment. The jig platform should be set in a rigid and accurate way that its features would not affect the accuracy in the trial assembly and final adjustment of sectional segments, consisting of complex configuration of leg if any, main body with front and rear cable stay structures.
- Keep the temperature changing in an acceptable range to constantly maintain the precision in fabrication of the pylon.
- Due to the huge size of the pylon segment with heavy weight, to avoid any deformation or prevent any damage done to the machined contact surface of the segment and the structure itself, during transportation.
- Geometrical alignment of pylon segment together with the site welding control.
- Control in painting quality.

12.6.1.2 Geometric Control during Erection

Cable stayed bridges are usually highly unstable structure during erection. This demands a well-coordinated and calculated step by step erection procedure including rigging method statements for pylon erection.

The main design issues to be addressed in order to erect the pylon are:

- How to stabilize the pylon at its base during construction;
- How to recover structure deformation during erection;
- How to install large, heavy box girder segments that had to be placed, one above the other, in a position especially when there are double inclination;
- How to manage the temperature effect, together with the tight tolerance of a bolted structure.

A detailed stage-by-stage analysis has to be done to determine stress distributions in the structure for all intermediate stages and to perform structural adequacy checks for erection stage. Sufficient strengthening of permanent structure has to be provided based on construction stage analysis wherever required.

A careful monitoring of the geometry throughout construction is required to ensure that the structure behaved as predicted and the target geometry is met. Since many of the quantities involved in the construction are only approximate values (exact elements dimensions and weight) or can be measured with a given tolerance (stay cable lengths and forces) and environmental conditions change during construction, and the calculation model itself is only an approximation of the real structure, a continuous monitoring of structural deformation during the construction is of fundamental importance.

Temperature changes, mainly during the day due to direct sunlight, modifies the real/actual geometry of the structure, continuously changing the position of working points making it difficult to install the ~~pipe~~ elements in their theoretical position. The movements also occur due to stressing and construction effects in addition to environmental effects as above.

Survey data has to be collected and examined for all the relevant erection stages, based on survey points which are marked during trial assembly. The survey points at deck and pylons have to be established with the aim of identifying the movements independent from installation of new elements and cable stressing and separate environmental actions effects (temperature, sun’s radiation, wind) from construction effects.

CHAPTER - 13

INSPECTION, MAINTENANCE AND STRUCTURAL HEALTH MONITORING

13.1 Introduction

Performance of cable stayed bridges during design service life shall be achieved by implementing maintenance management plan drafted during the design stage.

The Main Tension Elements (MTE) within a cable bundle are, in most cases, hidden from the view of inspectors. Access to cables for visual inspections or non-destructive testing (NDT) is generally difficult, and in the case of the anchorage zones, nearly impossible. Those who are tasked with inspection and maintenance of stay cables are faced with challenges for which proven and accepted methodologies and tools are limited and, in many cases, very costly.

Though IRC:SP:35 and IRC:SP:18 can be referred for inspection and maintenance of substructure, foundation and other auxiliary components, this Chapter focuses on deck, stay cable, and tower inspection, maintenance and SHM of cable stayed bridges. This includes the overall frame work of SHM, design of SHM system, the parameters that can be monitored and the corresponding sensors, data management and interpretation of results. SHM may be used to verify the design assumptions, to evaluate the performance of the bridge during its service, especially under extreme events, and when exposed to various environmental conditions, and, for maintenance management.

Annexure A5 gives guidelines for the Organisation Setup and Training Requirements for Inspection and Maintenance.

13.2 Inspection

Regardless of quality of design and construction it is inevitable that a bridge structure will suffer some degradation due to the effects of age, environment and loading that could often exceed the specified design values. The extent of this degradation is also influenced by the extent of the inspection and maintenance programme. The design life of long span bridges is generally achieved by specifying durable components, detailing for easy inspection access, having regular inspection and maintenance performed, and replacing components that have reached their service lives. Therefore, inspection access shall be an integral part of design.

Adequate space should be provided to allow access for construction, inspection and maintenance personnel. The space should also allow for material and equipment to be delivered from the deck or ground level to the cable anchors for repair and replacement work. Generally, appropriately sized access openings and interior lighting or a power source should be provided. The design should provide sufficient points of access and minimum opening sizes for inspection and maintenance.

13.2.1 *Scheduling of Inspections*

Different type of inspections and frequency as given in **Table 13.1** may be used if not incorporated in the operation and maintenance manual. **Table 13.2** gives the inspection requirement component-wise.

Table 13.1 Scheduling of Inspection

Types of Inspection	Inspection goals	Frequency
Initial inspections	First inspection performed and inventorized before the bridge is first opened to traffic to identify whether there is a change or update in inspection procedure for future maintenance.	One time
Routine Inspections	Visual inspection of the components given in **Table 13.2**.	Initially, the periodicity can be once in a month for six months. Subsequently the duration between two inspections can be increased to 6 months, depending on the condition of bridge component.
Detailed inspections	This should be a visual inspection from a close distance, NDTE can be used to identify cause of distress (see **Table 13.2**)	Once in three years/ or when major distress observed during the routine inspection.
Special Inspection/ Emergency Inspections	Inspections carried out immediately after earthquake, cyclone, flood, fire, impact damage etc. Can be done by foot with naked eye or with binoculars and with the help of appropriate NDTE (See **Table 13.2**)	After natural /manmade disasters such as earthquake, flood, cyclone, fire etc

Table 13.2: Type of Inspections and Parameters ~~to be Checked~~

Bridge component	What to inspect/measure	Inspection type				
		Regular		Special/Extraordinary After		
		Routine	Detailed	EQ	cyclone	other
Deck	Deflection/level	▲	√	▲	▲	▲
	crack	√	√	√	√	√
	vibration	▲	√	▲	▲	▲
Tower	crack	√	√	√	√	√
	tilt/inclination	▲	√	√	▲	√
Stay cables	vibration	▲	√	▲	√	▲
	tension	▲		▲	▲	▲
Cable pipe	Damage Deformation Discolouration	√	√	√	√	▲
Cable Anchorage	Damage, deformation, corrosion	√	√	√	√	√
Vibration control devices	Damage Deformation Rust or deterioration Leakage of viscous material	√	√	√	√	▲

√ Perform the inspection, ▲ Perform inspection if needed

13.2.2 *Inspection Program*

A typical inspection program of cable stayed bridges includes but is not limited to the following:

i) Change in structural geometry

ii) Deflections due to dead load and live loads

iii) Inspection for unusual cracks or deformations especially in concrete components subjected to high stress

iv) Distress/damage of expansion joints and uneven road level on either side of joint

v) Evidence of corrosion to reinforcement, other embedded steel and prestress / stay cable

vi) Erosion and abrasion of the substructure due to fast moving particles / objects in the river

vii) Collision damage

viii) Fire damage

ix) Deterioration of concrete due to chemical agents

x) Integrity of corrosion protection

xi) Scouring of foundation

The design reports along with “As Built Drawings” and construction records, form the basic data, for maintenance during operation stage. Therefore, it is essential to store these valuable data for a long period.

It is very much necessary to measure and record the initial inclination of tower and cable tension during the initial inspection of bridge, these form the basis to identify the abnormality of these components. It would be appropriate to perform the maintenance rationally and efficiently by creating a database of various records, to make it possible, to refer updated records of repair, inspection and monitoring at any time.

These guidelines will be used for inspection and maintenance of all components of cable stayed bridge superstructure. For the stay cable system, for recording the findings during inspection, the checklist given in **Annexure A6** shall be used.

13.3 Maintenance Aspects

13.3.1 *Maintenance issues to be considered during design*

Maintenance issues required to be addressed during the design stage are as follows-

i) Identify the design details that give rise to structural and durability problems,

ii) Design details that cause difficulties for inspection

iii) Provisions of access for inspection and (i) Inspection traveller(s) /underslung traveller/ maintenance carrier if required in case mobile bridge inspection cannot be used for bridge inspection.

13.3.2 *Maintenance Manual*

Contract documents shall specify the need for a maintenance manual. A maintenance manual shall be seen as an integral part of the design, as regular inspection and maintenance is required to achieve the recommended minimum design life of 100 years

The designer of a cable stayed bridge shall prepare a manual providing information about the basic demands on the components used in the bridge, the construction of the bridge, and suggestions about the inspection procedures of the bridge.

This Maintenance Manual shall be prepared after the completion of construction. Based on the design and construction, key areas for inspection need to be identified and the exact period between inspections for each component shall be included in the Manual. The Maintenance manual shall be approved by the client.

Special components such as the friction pendulum bearings and modular expansion joints will have a shorter cycle time than the main bridge structure due to their susceptibility to damage.

The maintenance manual shall contain at least the following:

i) Reference to the Design Documents (inclusive of the Corrosion Protection Plan)
ii) Reference to the As-Built Records
iii) Provide the specific time table and procedures for inspection, testing, maintaining and record keeping for the non-replaceable bridge components
iv) Provide the specific time table and procedures for inspection, testing, maintaining, replacing and record keeping for the replaceable bridge components
v) Indicate the qualification of inspection and maintenance team required for each activity
vi) Provide operational instructions and specifications of all mechanical and electrical equipment, if any

13.3.3 *Minimum Maintenance Requirements*

i) The maintenance team needs to have a full set of the drawings and maintenance schedules that indicate the frequency of inspection and maintenance for each part of the structure.

ii) The maintenance plans should also highlight areas where something has gone wrong during construction and which need special monitoring.

iii) For maintenance, repair, rehabilitation and strengthening of various concrete components of cable stayed bridge IRC:SP:40 shall be referred.

iv) The geometry of the cable stayed bridge shall be surveyed at regular intervals, and particularly after a major event, such as, a strong wind storm, an earthquake, deep scour due a major flood etc. Anomalies in the geometry, if identified in a reliable survey, tend to reveal distress or defects which shall be thoroughly investigated in order that remedial actions can be formulated and taken.

(iv) Where scour poses a significant risk to the structure, a scour monitoring plan should be developed as a part of the bridge inspection and maintenance program. Inspection requirements, including method and frequencies, should be specified.

(iv) For maintenance, repair and replacement of stay cable system, guidance from stay cable manufacturer/supplier shall be sought.

13.3.4 *Bridge Management System*

A comprehensive scientific bridge management system consists various tasks related to collection of data for inventory, to period of inspection, details of inspection of each component and procedures to monitor, analyze and implement the activities related to maintenance works. Authorities can use the bridge management system to achieve optimum use of available funds, adaptation of bridges to traffic needs and public safety.

Data Base Module

A pre-requisite for attempting any scientific system of management of bridge maintenance is to have a proper data base module comprising inventory, inspection records and maintenance, repair and rehabilitation data.

Inventory

The inventory module shall consist of records of (i) administrative data (ii) general data (iii) technical data (iv) geometric data (v) hydraulic data (vi) geotechnical data (vii) environmental data (viii) structural design data (ix) structural drawings and (x) photographs of bridge and various components before the bridge is open to traffic.

Bridge inspection record

Bridge inspection record consists of data gathered and recorded while checking the condition of various spans and following up the deterioration process from one inspection to another. This could include (i) initial inspection report, (ii) routine inspection report, (iii) detailed inspection report, and (iv) special inspection and or emergency inspection report.

Bridge maintenance, repair and rehabilitation data file (MR&R)

This module is the collection of all the data pertaining to the details of each component of Bridge/Viaduct, its inspection, its condition etc. In addition to the basic design data and construction data, it contains maintenance, repairs or improvement work data including as-build drawings, major repairs, and exceptional events such as earthquakes, abnormal floods etc. The data as stored provides adequate information to make decisions for repair & strengthening works.

13.4 Structural Health Monitoring (SHM)

13.4.1 *Brief of Existing Guidelines for SHM*

There are many existing guidelines for SHM of structures in different countries like Canada, UK, European Union, China, Australia, Switzerland and USA developed during the last two decades. Since the first structural health monitoring (SHM) guide, Guidelines for SHM **[1]**, was released, new SHM standards have been added around the world. The Canadian intelligent sensing for innovative structures, which published SHM of Innovative Canadian

Civil Engineering Structures, introduced basic techniques in SHM **[2]**. It also covers topics including repairing and strengthening of structures (use of fiber-reinforced polymers), intelligent sensing (integrated fiber optic sensors, fiber bragg grating sensors), economic significance (service life of structures will be extended with SHM), remote monitoring, and intelligent processing **[2]**.

The European Union developed the structural assessment, monitoring, and control (SAMCO) association for the maintenance of infrastructure and disaster prevention **[3]**. The SAMCO database is mainly used for infrastructure maintenance, and especially for bridge management. The SHM in the UK is applied to different civil infrastructures (e.g., dams, bridges, offshore installations, nuclear installations, etc.). In the UK, dams were the first structures for which legislation mandated SHM inspections, though recently more attention has focused on bridges **[5]**. Recommendations for SHM implementation in the UK include low-cost system (investment) and high benefit (return). The implementation of sensors, i.e., the total quantity of sensors that will be put on a structure, their positioning, the distance between them, and the algorithms and processing should be appropriately determined beforehand to avoid data overloads, including number of measurements, technical requirements, and different levels of accuracy for different sets of information collected. Environmental effects, including noise, on SHM systems could seriously influence or inhibit detection. In conclusion, the development of SHM in Europe provides good ideas and instructions for future SHM standards in other countries.

The rapid emergence of new standards and codes related to civil engineering in China has also led to the development of new SHM regulations **[4]**. More specifically, in 2016 the Chinese Ministry of Transportation published the first governmental SHM code regulating the installation of smart sensors in bridges **[6]**. China's ability to develop and implement new civil engineering designs, while rapidly increasing the size and volume of structures, can serve as a model for transportation agencies in other countries in writing, approving, and implementing their own new SHM regulations **[7]**.

Although the SHM standards from these institutions might not be enforced by their governments, the standards still give regulatory instructions for the implementation of SHM, which is supportive toward SHM code development.

13.4.2 *Need for SHM*

Structural health monitoring is a non-destructive in-situ structural sensing and evaluation method that uses a variety of sensors attached to, or embedded in, a structure to monitor the structural response, to analyze the structural characteristics for the purpose of estimating the severity of damage/deterioration and evaluating the consequences thereof on the structure in terms of response, capacity, and service-life **[2,8,9].**

Some key issues for cable stayed bridges that warrant the consideration of health monitoring systems include:

- Towers and cable elements of long-span bridges make the structure susceptible to the uncertainties of wind and its aerodynamic effects.
- Complex or proprietary devices used on long-span bridges often require monitoring for optimal performance. They are often used to mitigate between competing

demands on flexible long-span structures, i.e. the need to make the structural system rigid for dynamic effects such as wind, and flexible for others, such as seismic effects and differential settlements. High performance devices, such as lock-up devices, isolation bearings, tuned mass dampers, etc., are often needed to mitigate extraneous construction costs.

- Present design trend includes consideration of the high cost of total bridge replacement, and the trend to extend design life through the use of durable high-performance materials in conjunction with replaceable/sacrificial elements.
- Currently, the cost of inspection and maintenance of long-span bridges is relatively high.
- Consideration should be given to the nonlinearity and variability of parameters associated with the design of long-span bridges

SHM can be classified as short-term and long-term monitoring and the situations in which they are adopted is given below.

Short-term monitoring consists of live-load and dynamic testing. Data is only collected over a relatively short period of time when these field tests are taking place. The collected measurements can provide information about structural performance such as load and moment distributions, stress/strain levels, and serviceability issues. In addition, the collected data will provide valuable insights for future designs, and improve qualitative assessments of the bridge structures with different load tests.

Long-term monitoring usually consists of collecting ambient vibration and environmental data to evaluate structural condition over long periods of time. For instance, with long-term monitoring, changes or abnormalities in structural behavior can be detected without human interactions, or environmental effects (corrosion monitoring, seismic effects) on the performance of the structure can be quantified **[10,11].**

13.4.3 *Objective*

The main objectives of structural health monitoring include any/or all of the following:

i) design verification (short-term monitoring)

ii) traffic management (long-term monitoring)

iii) inspection and structural maintenance (long-term monitoring)

SHM is generally used to verify the design assumptions, to evaluate the performance of the bridge during construction as well as service, especially under extreme events, and when exposed to various environmental and loading conditions. This may be used for the maintenance and /or traffic management of bridges.

Where design life is predicated on minimum condition and performance of specific bridge components, such components shall be considered for instrumentation during the design stage.

13.4.4 *Overall Design Requirements including Institutional Set Up*

The complete design process of a SHMS can be summarized by following these steps:

1) Identify the need for monitoring
2) Set the objectives of the monitoring system
3) Undertake risk, uncertainty and opportunity analysis
4) Identify parameters and response requirements
5) Select appropriate sensors
6) Select sensor location and communication system
7) Install system and calibrate
8) Acquire data, process, manage and report
9) Assess data/study of report
10) Decide on the need for remedial action

The design process covers several traditional study fields, such as civil engineering, electrical engineering and computer science. Therefore, the Institute or the organisation undertaking the design and implementation of SHM should have bridge experts, electrical, electronics and communication and computer engineers. They should have trained staff for installation and calibration of sensors. They should have proven track record in design, installation and performance monitoring of at least two cable stayed bridges.

Parameters, Location and Number of Spans to be Monitored

While designing a SHM system, first step is to decide whether short-term or long-term monitoring is required as discussed in **Section 13.4.2**.

The structural response parameters to be monitored include strain, acceleration and displacement of deck at critical locations, cable forces, strain and displacement of tower tip.

The environmental parameters include wind intensity and direction, water level, atmospheric temperature and humidity.

There are other loadings/ effect of actions to be monitored such as vehicular loading (traffic volume, type of vehicle, speed of vehicle, axle loading of trucks, axle spacing, distance between trucks) pedestrian loading/ crowd loading, thermal effects, seismic effects and wind effects.

Selection of location of sensors shall be done based on the three-dimensional analysis of cable stayed bridge under various loadings. It may be appropriate to identify the critical locations for performance monitoring to be identified by the structural designers and included in the structural drawing.

The minimum number of spans to be instrumented and monitored shall be one unit (between two expansion joints) if the purpose of SHM is to verify the design assumptions and to collect a baseline performance data.

All cables stayed bridges with main span length more than 300 m shall be selected for performance monitoring.

In the case of multi-span cable stayed bridge, with identical spans, at least one cable stayed unit shall be considered for SHM.

Cable stayed bridges provided with seismic isolation or vibration control devices shall be instrumented to monitor the performance of bridge

For more details on design and functional requirements of SHM, SHM frame work and safety evaluation of bridge using the data collected, **Annexure A7**.

13.5 Case Studies

This section gives examples of a few bridges which have been instrumented for Structural Health Monitoring purposes. These examples are given so as to provide typical arrangements of instrumentation. It is expected that when any particular bridge is considered for being instrumented, the exercise given in **Section. 14.4.4** will be carefully carried out in order to optimise the arrangement for its economy and effectiveness.

13.5.1 *Signature Bridge in Delhi* ***[12]***

Bridge Details

The Signature Bridge is an eight-lane with semi-harp shaped cable-stayed road bridge across the Yamuna River, Delhi. It has an overall length of 575 m, and a main span of 251 m. The composite deck contains three main girders, with cross girders at 4.5 m spacing. The polygonal shaped tower provides, to a substantial extent, the stress balance required to support the deck. It also contains lifts and stairs providing access to the top of the 151-meter tower, for maintenance and sightseeing purposes.

Parameters to be Monitored and Location Selection

The following data are recorded by the system and the sensor layout is shown in **Fig. 13.1** and summarized in **Table 13.3**

Environmental Factors

Environmental conditions at the top of the tower are recorded by a weather station which measures the ambient temperature, relative humidity, precipitation levels, barometric pressure, as well as wind speed and direction. Wind speed and direction is measured by an ultrasonic sensor at the deck level. Relative humidity and temperature inside box section of the deck structure is measured by four hygrometers. The level of corrosion in the tower base structure is monitored by means of embedded chloride ion and corrosion sensors. In addition, seismic events, which are expected to have a serious impact on the entire structure, would be measured at the tower base in each leg plinth using 3D high-definition accelerometers.

Load Factors

Natural vibrations are measured by eight 3D-accelerometers, which will be capable of relocation along the length of the structure to provide total coverage on a cyclical basis. Traffic data is recorded by laser analysers, enabling lane usage, average lane speed and vehicle classification are to be determined. Traffic congestion, accidents and other anomalies will be captured by a video surveillance unit featuring colour cameras, capable of recording 30 frames per second and night-time usage.

Bridge Structural Response

Strain in the tower, compensated for temperature, is measured by welded strain gauges, while strain in the deck girders, also temperature compensated, is measured by bolted strain gauges. Stay cable forces is measured by electromagnetic sensors, weatherproof and telecommunications shielded, on the longest, shortest and medium length cables of each set of cables – two sets on the main span and one on the back span. Cable vibrations are recorded by 3D-accelerometers, which is detachable to allow relocation during and after the construction phase to assist in calculation and verification of damping, as well as assessment of changes in harmonics during operation. Tilt of the deck on each span and of the pylon (above and below the elbow) is measured by inclinometers, and displacement of the deck by strain gauges at the expansion joints.

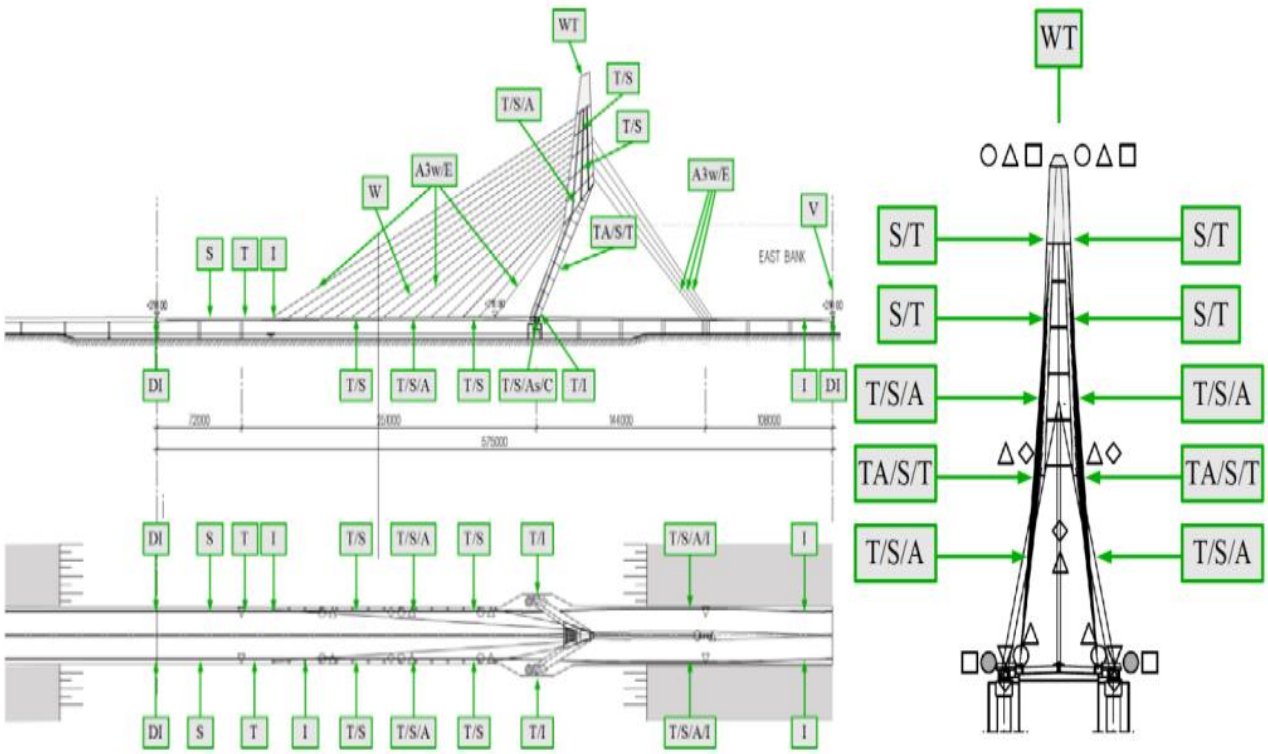


Fig. 13.1 Sensor Layout of Signature Bridge

Data Acquisition, Transmission, Processing and Management

The data acquisition unit is based on industrial graded cPCI systems with dual-core CPUs, running data acquisition and data calculating software. The unit controls all sensor functions and provides all measured data with source codes (name and position of sensor) and time stamps to allow accurate time synchronisation. A/D-conversion is done by multi-channel, 24-bit converter modules, located as closely to the sensors as possible. All raw data measured by the sensors is stored on two independent hard discs to prevent data loss in case of disc failure. Pre-processing of the monitoring data also takes place here. The master station is equipped with independent time sources (GPS) to ensure self-sufficiency should connection to the local network be lost. This ensures preservation of records during critical events such as traffic accidents or earthquakes. The unit is also protected by an air-conditioned cabin to enhance reliability and durability.

Table 13.3 Summary of sensors included in the SHM system

Sensor	Logo	Sensors according to Specifications	Channels according to Sensors
		Sensors	Channels
Structural temperature	T	20	24
Strain gauge boltable	S	10	10
Strain gauge weldable/ glueable	S	10	10
Wind sensor 3D	W	1	4
Wind multi-sensor (Wind 2D direction and speed, ambient temperature, precipitation levels, barometric pressure, relative humidity)	WT	1	6
3D accelerometers, cables	A3w	18	54
3D accelerometer, deck & pylon	A	8	24
Seismic accelerometers	As	4	12
Displacement sensor	DI	4	4
Inclination, tilt	I	4	8
Corrosion	C	3	6
DV Camera	V	4	
Traffic analyzer	TA	8	
Electromagnetic sensors	E	9	9
Sum		104	171

The data connection between the master station on the bridge and the bridge operator's control room is by means of a wireless LAN bridge using a private static IP as described in **Fig.13.2.** This can operate over a distance of four to five kilometres. This system enables real-time data transmission, independent of mobile telecommunication service providers.

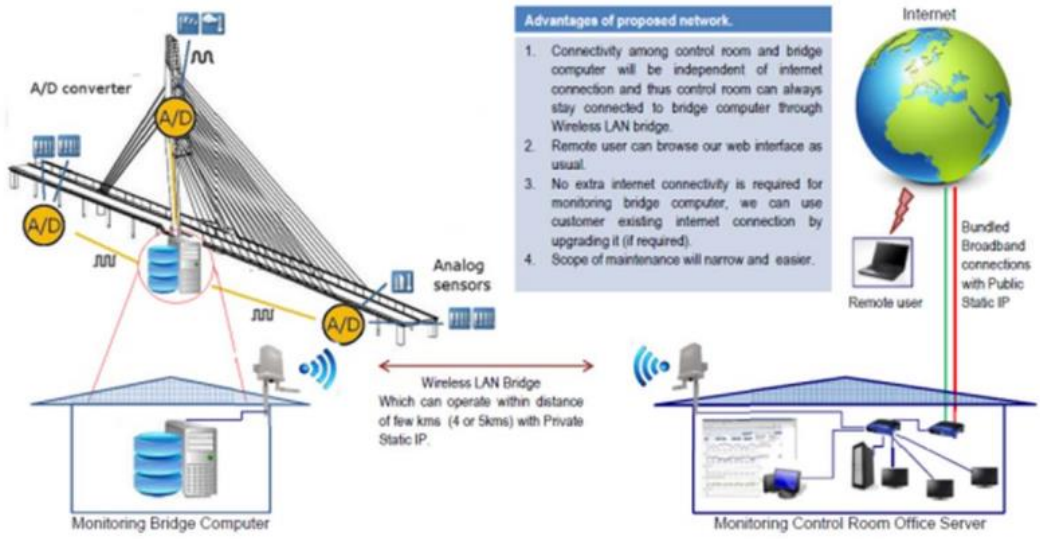


Fig. 13.2 Data transmission

Evaluation of Structural Performance

Data presentation is controlled by a local monitoring data server in an office of the client's choice. Since the server is connected to the internet and the data transfer from the master station on the bridge to the server is wireless, the location can be freely chosen. All data received from the master station is stored permanently on two independent hard discs to prevent data loss in case of disc failure. The server is also capable of processing, managing and presenting data from any other permanent monitoring system that might be connected in the future.

The user interface enables an authorised user to log on to the system, and view any data which may be of interest, for any period of time during which measurements were made. It also enables the user to input condition and event values and, if necessary, instructions for personnel. Information is presented in a user-friendly and simply understandable format. High frequency data is displayed in clear graphics (e.g. mimic panel format). The user can also instruct the system to provide an alarm if predefined threshold values of any variable are exceeded. Such alarms can take the form of a visual notification on a monitor, or an email or SMS to a selected person. This user interface is accessible at any time, from anywhere in the world with an internet connection.

13.5.2 SHM description of a Steel Cable Stayed Bridge, Jindo Bridges, South Korea [13] (Fig 13.3)

Bridge Details

The Jindo Bridges are twin cable-stayed bridges. Each of these bridges consists of three continuous spans, with a 344 - m central main span and two 70 - m side spans. The width of the first bridge is 11.7 - m with the design traffic velocity of 60 km/hr and the width of the second bridge is 12.55 m with traffic design velocity of 70 km/hr. It has a streamlined steel box girder supported by 60 high-strength steel cables connected to two towers.

Parameters to be Monitored and Location Selection

The first Jindo Bridge has 38 strain gages, four inclinometers, two anemometers, two seismic accelerometers, five uniaxial capacitive accelerometers, and 15 uniaxial piezoelectric accelerometers. The second Jindo Bridge has 15 thermometers, 15 strain gages, four biaxial inclinometers, two string pots, two laser displacement meters, 24 Fiber Bragg Grating sensors, 20 uniaxial capacitive accelerometers, two biaxial force balance type accelerometers, and three triaxial seismic accelerometers.

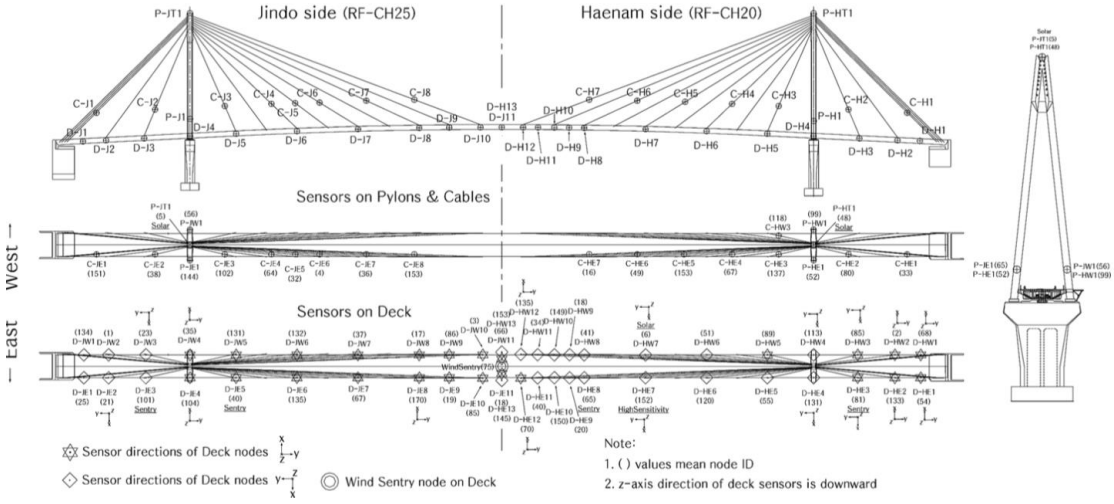


Fig. 13.3 Sensor location

A 3-D ultra-sonic anemometer shown in **Fig. 13.4** has been incorporated into the Wireless Sensor Network (WSN) for reliable wind monitoring of the Jindo Bridge. Because of its high resolution (wind speed: 0.01 m/s, wind direction: 0.1 degree) and good accuracy (wind speed: ±1%, wind direction: ±2 degrees).

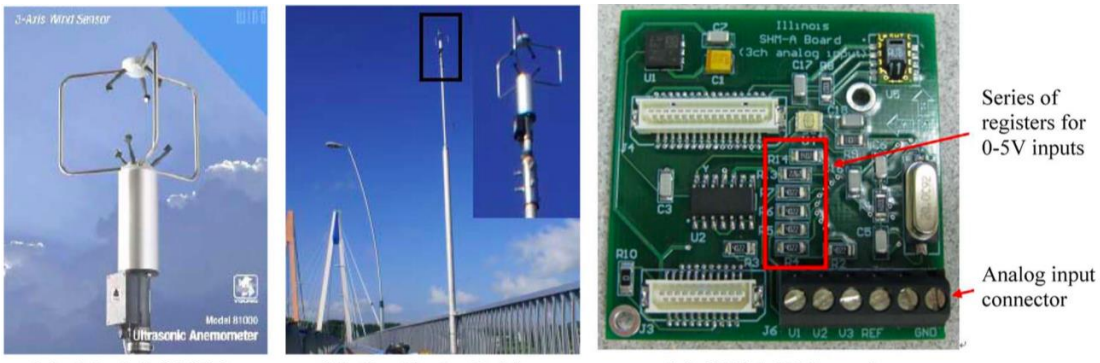


Fig. 13.4 3D ultra-sonic anemometer and SHM-W board

Data Acquisition, Transmission, Processing and Management

The key features of these services include Remote Sensing, a remote data measurement application; Decentralized Data Aggretation, data measurement and on-board computation to calculate the correlation functions for decentralized local groups; imote2comm and

autocomm, an automatic terminal tool to interact between the base station PC and a gateway node; Vbat, a command for checking a leaf node's battery level; TestRadio, a tool for assessing radio communication quality, and many others.

The continuous and autonomous monitoring service critical for this deployment is AutoMonitor, an autonomous SHM network management application, which combines RemoteSensing, ThresholdSentry, and SnoozeAlarm (Rice and Spencer 2009). SnoozeAlarm is a strategy that allows the network to sleep most of the time, thus improving energy efficiency and allowing long-term system deployment. To wake the network for an important event, the ThresholdSentry application defines a specified number of the leaf nodes as sentry nodes. The sentry nodes wake up at predefined times and measure a short period of acceleration or wind data. When the measured data exceeds a pre-defined threshold, the sentry node sends an alarm to the gateway node, which subsequently wakes the entire network for synchronized data measurement. In this way, AutoMonitor enables the automatic, continuous monitoring with reduced power consumption.

The AutoMonitor application employs multiple threshold levels for the Jindo Bridge SHM system. When a single threshold level is used to define the maximum number of events during given period, if the threshold level is too high, the data measurements would rarely if ever occur. If too small a threshold level is used, the number of events can be exhausted with small structural responses, hence possible strong vibrations during remaining period cannot be captured. The multiple threshold levels, having separate maximum number of events, make it possible to measure both rare but strong responses and frequent but low-level ambient vibration during a given period, which enables more effective SHM.

Another feature of the AutoMonitor application is a wind threshold sentry. The wind sentry node is equipped with SHM-W sensor board with ultrasonic anemometer. Strong winds, such as those that occur during typhoons, induce large structural responses. Therefore, the wind sentry will trigger network sensing when the velocity exceeds a threshold. In this deployment, both vibration- and wind-based sentry nodes have been installed for the AutoMonitor application. In summary, the final software version is the AutoMonitor application having both data measurement and on-board computational functionality, a multiple threshold triggering strategy using vibration/wind sentries pursuing energy efficiency using SnoozeAlarm.

Table 13.4 shows the list of the network parameters and values for the initial state and the updated state.

13.5.3 *SHM description of Rion-Antirion Bridge, Greece **[14]***

Bridge Details

The Rion-Antirion Bridge is a 4-tower. 5 span cable-stayed structure. The continuous composite deck has total length of 2252 m, with 3 main spans of 560 m and side spans of 286 m, and is suspended by 4 concrete towers, with total height of 189 up to 227 m, through 368 cables with total length from 79 up to 295 m. At each far end of the deck, a steel rotating frame (RF) supports the structure allowing longitudinal movement that is accommodated by special designed expansion joint. Furthermore, at tower and RF

locations, the deck is transversally restrained through a fusing steel element that releases the deck when the transverse load, on each element, exceeds ±10.500/±3.400 kN (tower/RF).

Table 13.4 shows the list of the network parameters

Category	Parameter	Initial state	Updated state
RemoteSensing	Number of *RemoteSensing* events	1 per day	4 per day
	Time synchronization wait time	30 sec	30 sec
	SENSING START DELAY	15,000	50,000
	NODE SENSING START DELAY	1,000	1,500
	Sampling frequency	50 Hz	10 Hz
	Channels sampled	3	3
	Number of data points	1,000	5,000
Snooze Alarm	*SnoozeAlarm* wake/listen time	750 ms	750 ms
	SnoozeAlarm sleep time	15 sec	15 sec
	SnoozeAlarm duty cycle	4.76%	4.76%
ThresholdSentry	*ThresholdSentry* check interval	20 min	10 min
	ThresholdSentry sensing time	10 sec	10 sec
	Threshold value for vibration sentry	10 mg	10 mg, 50 mg
	Threshold value for wind sentry	N/A	3 m/sec, 8 m/sec
Watchdog Timer	Period	10 min	60 min
	Update	1 min	60 min

Parameters to be Monitored and Location Selection

The current architecture of the monitoring system, **Fig.13.5** can be divided into 4 different levels namely, (a) Sensors (b) Power supply and signal transferring (d) Digitalization, acquisition and signal processing and (d) Communication network and data management which are explained in detail in the upcoming sections. Largely, the monitoring system consists of:

i) Weather stations for monitoring of wind direction, velocity, ambient temperature, humidity and air pressure,

ii) load cell on fuse restrainer,

iii) accelerometers for monitoring wind effects on deck,

iv) accelerometers for characterization of earthquake motion,

v) accelerometers for monitoring earthquake effects on deck,

vi) accelerometers for monitoring earthquake effects on towers,

vii) monitoring of expansion joint openings,

viii)detection of water inside tower legs,

ix) road temperature,

x) characterization of wind effects on stay cables:

- accelerometers for monitoring vibrations induced by traffic and wind,
- mono strand load cells for monitoring of tension in the stays,

- displacement transducers for monitoring of the vibration amplitudes,
- strain gauges for monitoring of internal stresses in gussets (connection between stay anchorage and deck)

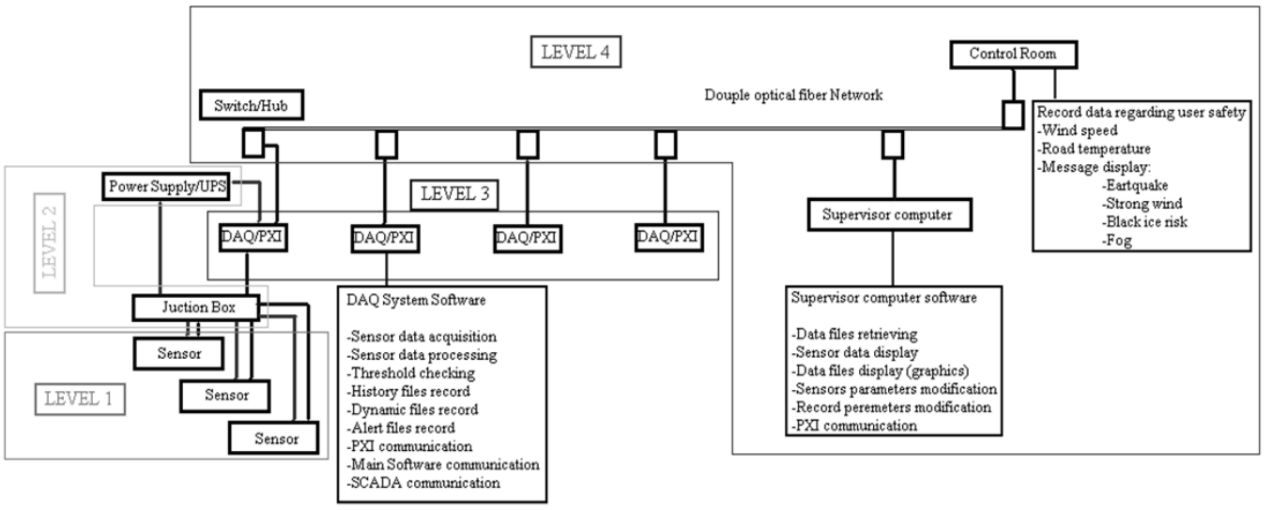


Fig. 13.5 Overview of structural monitoring system

Sensors:

A large variety of sensors as shown in **Fig. 13.6** and listed in **Table 13.5** were incorporated to the monitoring system in order to record the response of various structural elements under different loading conditions.

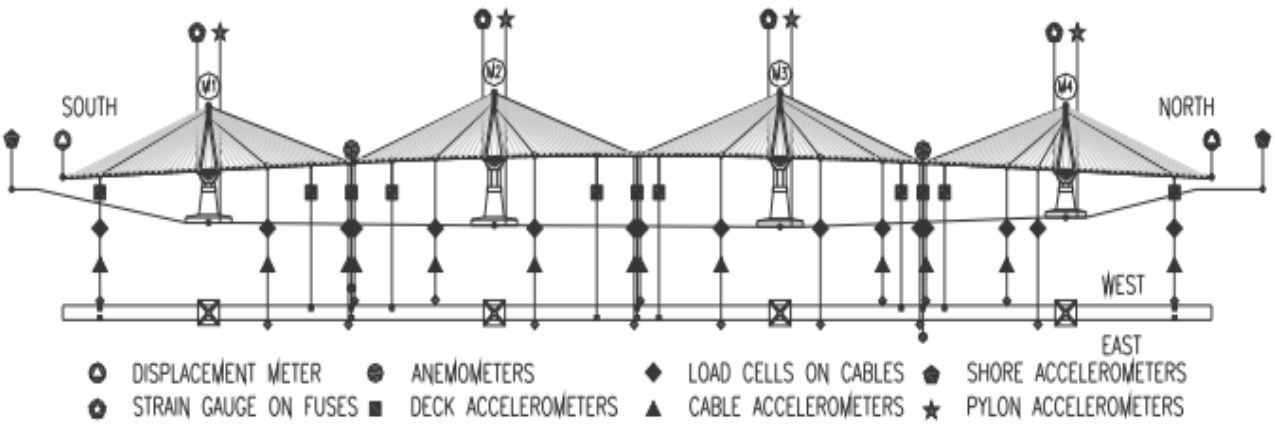


Fig. 13.6 Layout of Structural Monitoring System

Table 13.5 Sensor Description

Sensor	Quantity	Expected range of values	Actual sensor range	Monitored phenomenon
3D anemometers	2	0-50 m/sec	0-60 m/sec	Wind intensity
Temperature and Humidity sensor	2	50° C/0-100%RH	-50°C, up to 50°C/ 0-100% RH	Thermal loading
3D Pylon accelerometers	12	±1.9g (top) ±1.0g(base)	±20g(top) ±3g(base)	Pylon vibration (Earthquake/wind)
1D/3D Deck accelerometers	3/12	±2.7g	±3g	Deck vibration (Earthquake/wind)
3D Ground accelerometers	2	±0.48g	±3g	Earthquake
3D Cable accelerometers	13	-	±3g	Cable vibration Wind
Monostrand load of cables	16	0 up to 75% F_{GUTS} (199 kN)	0-320 kN	Cable load variation (Wind/Earthquake/Balance)
Magnetic distance meter	2	+1260/-1150 mm	3 m	Expansion joint opening (Earthquake/Balance/Thermal)
Strain gauges (full bridge)	4	±10500 kN	±1500$\mu\varepsilon$ ±17000 kN	Wind induced lateral load
Road temperature sensors	4	-	-50°C, up to 50°C	User safety (black ice risk)
Deck temperature sensors	5	-	-10°C, up to 80°C	Thermal loading

Data Acquisition, Transmission, Processing and Management

Due to the size of the monitored structure (more than 2500 m between extreme sensors), 4 different digitalization and acquisition units (DAQs) are installed, each on one tower. The DAQs are located in controlled temperature shelters and are specially designed to be operable in harsh environments. The following operations are performed in each DAQ:

- Low pass hardware filtering at 10 kHz
- Digitalization at 500 Hz
- Conversion of signal to engineering units
- Continuous threshold checking
- Data file creation and real time transmission of selected values

A significant aspect, necessary for any further analysis of the recorded signals, is synchronization. In current architecture all the DAQs are synchronized with a server through SNTP protocol.

The communication of each DAQ with the supervisor computer (SE) and with the Control Room of the Bridge is performed through an optic fiber network that is installed along the deck. Thus, the collected data are retrieved, evaluated and permanently stored to dedicated media inside the operation building. Additionally, through the SE, the end user can overview all the measurements (real time) and access/modify all record parameters (thresholds, scaling, acquisition parameters etc.) that can be adjusted to each monitored event.

Evaluation of Structural Performance

Two main categories of data files are created by the monitoring system:

a) History files (0.5 sec averaged values recorded every 30 sec, except wind speed and direction that are 10' average)

b) Dynamic files (High sampling frequency at 100 Hz with 60 sec duration): The History files are created continuously while the dynamic files are recorded every 2 hours (Automatic) or when particular threshold has been over passed (Alert files) or even at user demand (Request).

All types of files (History/Automatic/Alert/Request) are very useful in order to understand the actual bridge response. Each of them can be used for different analysis purposes. In particular, the History files are very useful in:

- Characterization of Environmental conditions, (wind, temperature)
- Estimation of deck creep and shrinkage
- Evaluation of static impact of wind loads
- Identification of potential loss of cable load from overall force distribution

13.5.4 *SHM Description of Chaibu Bridge, China* ***[15]***

Bridge Details

This is a two-tower cable-stayed bridge carrying six lanes of traffic over the Fuchun River. It is located on the highway between the Meirong and the Fengchuan in the Tonglu County, China. The construction of the bridge began in April 2014 and is expected to complete in December 2017. The cable stayed bridge has a total length of 412 m (106 m + 200 m + 106 m). There are 104 cables divided into four central cable plane parts and arranged in twin towers in total. The decks are composed of cast-in-situ concrete box girder by cantilever method. Each cable tower is a single column tower, and the main tower is 64 m high using box-section. The pier is double thin-wall structure with rectangle section. The connection of bridge's girder, tower, and pier is rigid joint.

Parameters to be Monitored and Location Selection

The whole SHMS consists of four subsystems, including sensor subsystem, data acquisition and transmission subsystem, database management subsystem, detection and evaluation subsystem.

Sensor Subsystem

The optical fiber strain sensors are used to monitor the changes of strain. Additionally, weigh-in-motion system (WIM), one-direction accelerometers (ACC), and global positioning system (GPS) are installed on the bridge to measure the load of different vehicles, acceleration and displacement signals of structure in time domain. In this monitoring system, AER is established to record the atmosphere temperature and humidity for gaining the change law of some physical parameters and revising the test data. The **Fig. 13.7** shows the details of sensors layout and the **Table 13.5** lists the number of sensors

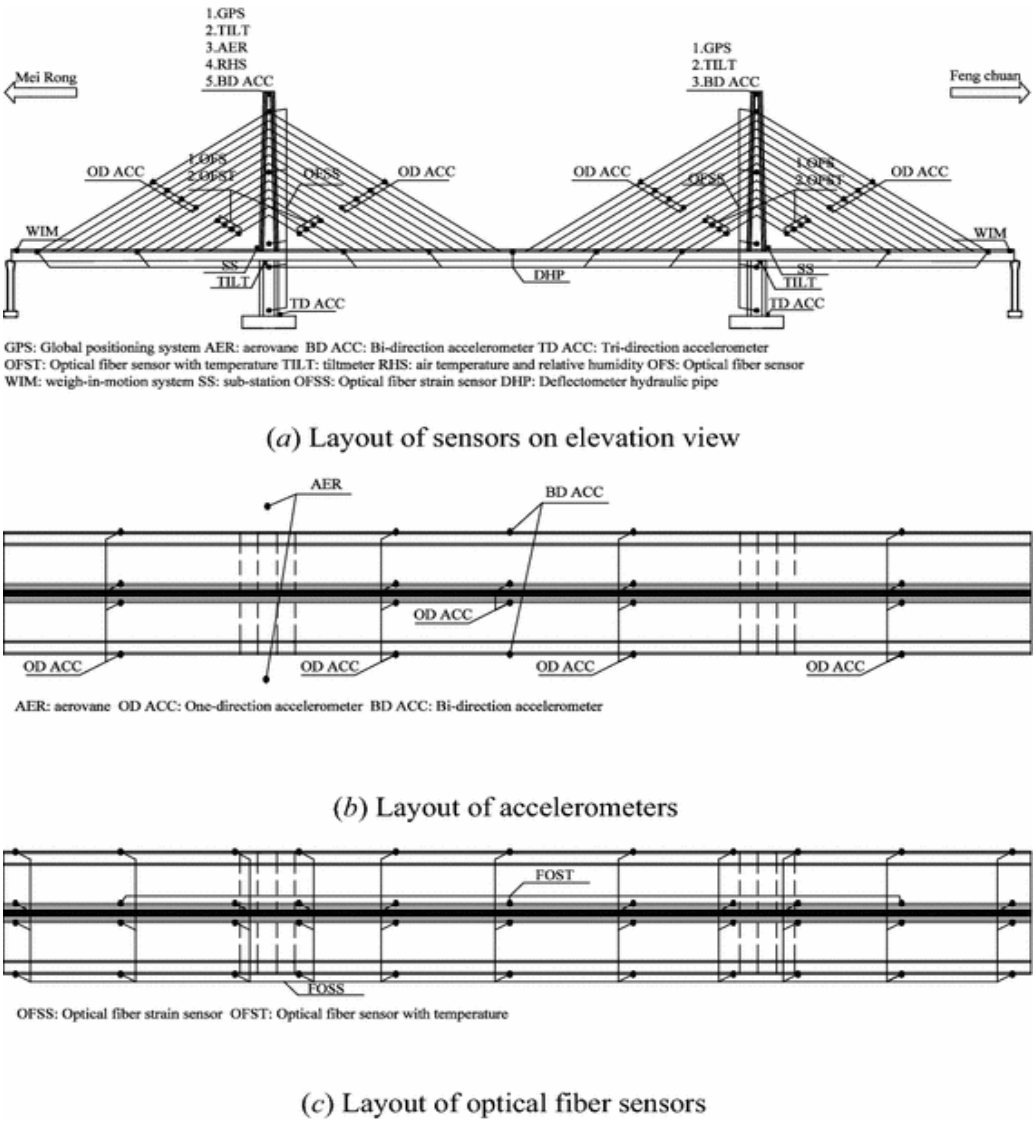


Fig. 13.7 Details of Sensors Layout

Data Acquisition and Transmission Subsystem

The monitoring signals were divided into static and dynamic signals by the frequency of signals. Based on distributed static signal with RS485 serial bus, the input/output (I/O) data acquisition system is designed. As for the high sampling rate dynamic sensors, it takes the IEEE1588 standard to ensure the acquisition system clock synchronization. The distributed data acquisition system uses industrial Ethernet for Dynamic signals transmission. Besides, LabVIEW, a product of NI Inc., USA, is selected to be the software to write the program for the collection of sensor signals. At present, the signal transmission technologies can be classified into two categories that are the wired transmission and the wireless transmission. In this SHMS, these two signal transmission technologies are combined for transferring. Firstly, the signals like OFSS and OFST are transmitted to the industrial computer installed on the bridge by wired transmission technologies. And then the data transmit to the server installed in the station building by wireless transmission technology.

Database Processing and Management Subsystem

A database based on the SQL Server 2008 is developed as a module of this SHMS. The entire database is composed of the static and dynamic database. The static database is used for storing static information, e.g. geography information of the bridge site, detailed design information of the bridge, layout of sensors. The dynamic database is related to the dynamic sampling rate. To prevent the shortage of the memory, some measures should be taken for database management when the memory comes to a threshold set by bridge manager. Meantime, computer-user interface designs for interactive requirements based on B/S model. Because the B/S model interface lowers the requirement of client configuration and improves systems operability. Some safety means are used in the SHMS to prevent intrusion of the database, such as password, virus resistance. The connection between different subsystems is shown in the **Fig. 13.8**

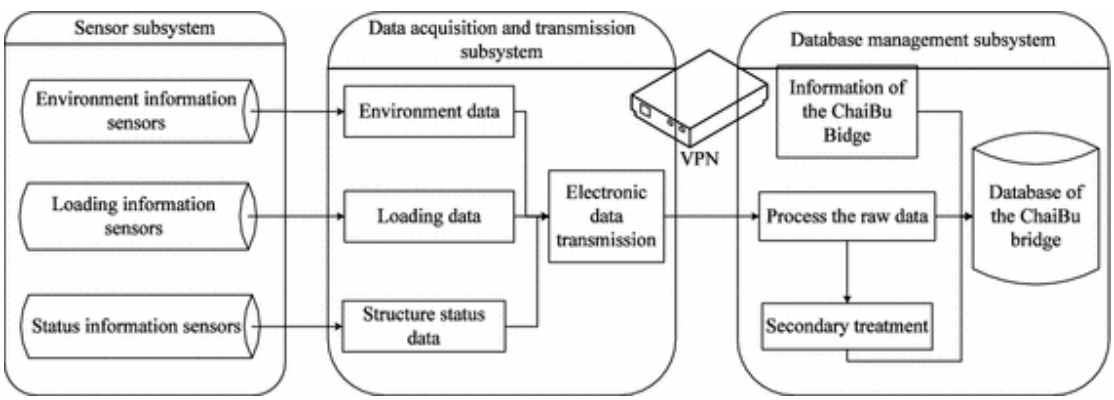


Fig. 13.8: The connections between different subsystems

Evaluation Subsystem of Structural Performance

The detection and evaluation subsystem in the SHMS is to detect damage and evaluate the condition of the bridge. The typical damage detection methods, such as frequency response model, modal curvature method, and data fusion technique for damage detection, are integrated into this subsystem and can be automatically operated by utilizing the

measured data. As for the FEM, the sensitivity analysis method is employed to automatically update the FEM. For example, material elastic modulus and pre-stressed force will change with time. With several iterative cycles, the FEM can be used for analyzing the structure condition

CHAPTER - 14

AESTHETIC CONSIDERATIONS

14.1 Introduction

The synthesis of structural system and visual form is near perfect in the case of Cable Stayed Bridges, which derive their aesthetics from their pleasing appearance, economy of material usage and lack of any embellishments for purposes other than purely structural. The flow of forces in the structure are self-evident and that is part of the enchanting appearance of this concept of bridge construction. All the members are predominantly subjected to axial forces, with the pylon and the deck under compression and the cables under tension which leads to a very efficient structural system. The low consumption of materials in the light superstructure results also in saving of materials and costs of the foundations thereby enhancing the sustainability aspects. Aesthetics of Cable stayed bridges require particular attention to its integration in urban spaces.

By their very nature the design and construction of Cable stayed bridges requires skills of a high caliber apart from an intimate knowledge of structural behavior. They are a popular option for long bridges when the main spans are upwards of 200 m, **Fig. 4.1 (Chapter 4)**. Due to its high degree of indeterminacy the use of high speed computing and appropriate softwares become a basic requirement for its analysis

14.2 Structural System and Structural Elements

Aesthetics of Cable Stayed Bridges is closely related to:

- Tower shape,
- Arrangement of stays
- Structural form of the deck
- Proportion of elements below the deck
- Integration with the surrounding environment
- Lighting

Aesthetics of cable stayed bridges must be seen differently as compared to conventional bridges. The additional elements of pylons and stay cables introduced in the structure have a huge visual impact. The size of the pylon with its enormous height becomes the most dominating feature of the bridge, and, most of the time even the landscape. The closely spaced stay cables provide a transparent veil that gives the bridge an element of delicacy. The appeal of the cable-stayed bridges is not limited to engineers but has evinced great interest of the public at large. The interest generated in the media for the Signature Bridge in Delhi, **Fig. 4.8 (Chapter 4)** is a case in point.