14.3 The Tower

The tower shape is dependent on a number of factors such as one/two planes of stays, the foundation size to which must be transferred all loads and the structural elements that keep the second order effects (buckling) under control. The straight towers of **Fig. 14.1**, Vidyasagar Setu, Kolkata and the 'cranked' towers of **Fig. 14.2** Atal Bridge @Basholi have acquired their shapes to suit the foundation size.

The horizontal struts connecting the two tower shafts provide relief from second order effects. The tower is under heavy compression as it has to balance the vertical component of the tensions of the stays. The inclination of the shafts constituting the towers is an important aspect of design. The two shafts can be kept separated, **Fig. 14.1**, or, fused together at some height and extended as a single vertical element forming an inverted Y shape, **Fig. 14.2**.



Fig. 14.1. Vidya Sagar Setu: Tower shape is governed by foundation size.

Note: The connecting beam at the top to reduce second order effects



Fig. 14.2. Atal Bridge @Basholi Tower shape is governed by foundation size

Note: the merging of towers towards the top to reduce second order effects

https://www.aboutpathankot.com/atal-setu/

14.4 Arrangement of stays

It is well recognized that the harp system of stay cable arrangement, **Chapter 3**, is the most preferable one from the point of view of aesthetics. Even for two planes of stays in a harp arrangement they do not intersect whatever be the position and angle of the viewer.

14.5 The Deck

The deck is usually very thin with a depth to main span ratio sometimes as low as 250 and this characteristic of sleekness contributes greatly to the overall aesthetics. The constant depth throughout the bridge length with closely spaced supports provided by the stay cables adds further value to the appearance.

14.6 Proportion of elements below the deck

From a purely aesthetic point of view, the proportion of the structure should be such that the pier height should be much smaller than the spans of the superstructure that it

supports. Due to the long spans involved in the cable stayed bridge constructions this requirement is fulfilled automatically. Also, in Indian Conditions there is rarely a requirement of big heights that are necessary for the introduction of clearances for ocean liners and big ships.

14.7 Integration with the surrounding environment

The Cable stayed bridge can have numerous variations of the basic bridge type which makes it easy for it to fit into most surroundings once there is a requirement of a long span. The integration of the bridge into its environment or landscape, or the urban or semi-urban situation remains the most important consideration in relation to aesthetic design, but it seldom necessitates any structural modifications An example that shows how the bridge has blended beautifully in the hilly terrain is shown in **Fig. 14.3** Atal Bridge @Basholi.

14.8 Lighting

Lighting of the cable-stayed bridge at night must form an integral part of the design. This is in addition to the lighting of the deck carriageway which is a purely functional requirement. The closely spaced stay cables are of great interest from the point of view of aesthetics as they provide a translucent background on which lighting can be projected with great effect. The light source for aesthetic lighting can be on the central verge of the deck, on the footpaths or at the top of the tower. These options and a combination of them provide ample opportunities for creativity as far as projection of lights on the stay cables is concerned. The lighting provided for aesthetic purposes should ensure that it does not create a glare for the drivers and also does not interfere with aircraft navigation.

Lighting of the cable stays with LED lights can greatly enhance the appearance of the bridge at night. Excellent example of this can be seen in **Fig. 14.4** (Chambal Bridge in Kota) and **Fig. 14.5** (Mandovi Bridge, Goa). They have made a new tradition called “structural art”. Superimposing laser images on the stays is another area of expertise. The images can be changed from time to time.

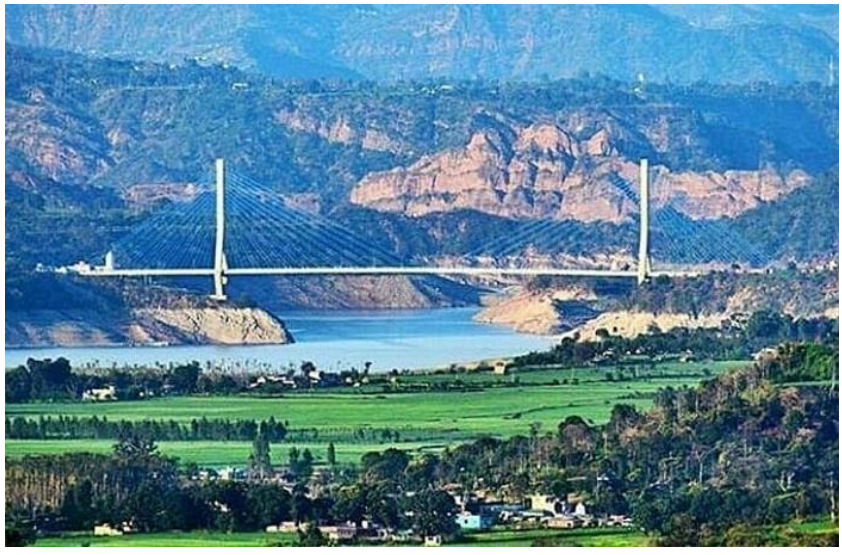


Fig. 14.3 Atal Bridge @Basholi. Aesthetic blending into the hilly terrain



Photo: Om Birla/Twitter

Fig. 14.4 Chambal Bridge in Kota



Fig. 14.5 Mandovi Bridge, Goa

IRC:SP:136-2023

Annexure A1

List of Cable Stayed Bridgesc in India (Pertaining to Chapters 1 & 3)

No.	Bridge Name	Owner	Completion Year	Location in India	No of Traffic Lanes	CSB Bridge Length (m)	Span Arrangement (m)	Width (m)		Number of Cable Planes	Girder Support at Pylon (Integral / Bearings/ No support)	Cross Section of Girder (Concrete / Composit)	Girder Depth (m)		Tower Height above Bridge Deck level (m)	Tower Height above Foundation (m)	Stay Cable Arrangement		Broad Quantities				
								Total width	Carriageway width				at Pier	Mid. Span			Shape, Nos. of Plane	Cable Interval at Deck (m)	Conc per sqm of Deck Area (cum)	Str Steel per Sqm of Deck Area (T)	Stay Cable per sqm of Deck Area (T)	Prestress per Sqm of Deck Area (T)	Reinf per Sqm of Deck Area (T)
100	Basohli Bridge, Jammu	Border Roads Organisation	2015	Pathankot		592	121 + 350 + 121																
101	Mandovi Bridge, Goa	GSIDCL, Goa	Ongoing	Goa	3 + 3	620	85 + 3x150 + 85	21	17	1	Integral	Concrete	3.00	3.00	35.0	66	I-Shape,1	6	0.545	NA	0.017	0.024	0.111
102	Beyt Dwarka Bridge, Gujarat	MORTH	Ongoing	Gujarat		900	200 + 500 + 200					Composite											
103	2nd Hoogly River Bridge, Kolkata	Kolkata Port Trust	1992	Kolkata		823	183 + 457 + 183					Composite											
104	Bardhan Bridge, West Bengal	West Bengal Govt	2019	Bardhman		188	64 + 124																
105	Rohini Amusement Park, Delhi	Unitech	2010	Delhi		63.8	16.9 + 30 + 16.9																
106	Pedestrian Bridge, Mahatma Mandir	R & B, Gujarat	2015	Gandhinagar, Gujarat	N.A.	174	74 + 100	8	6.4	2	Bearings	Composite	1.85	1.85	63.5	63.5+9	Harp, Two	12.5					
107	Skywalk Navi Mumbai	CIDCO	2012	Navi Mumbai	N.A.	121	70 + 51	6.5	6.5	2	Bearings	Steel	4.00	4.00	41.0	45	Fan Two	6					
108	Zuari River bridge, NH17 / NH66, Goa	PWD, Govt of Goa	Ongoing	Goa	4	640	140 + 360 + 140	36	36	2	Bearing	Composite	2.00	2.00	86.5	100	FAN, 2	12	0.225	0.274	0.03472	NA	0.125
109	Tiracol river bridge, Goa	GSIDCL, Goa	Ongoing	Goa	2	350	70 + 210 + 70	12.6	12.6	2	Bearing	Concrete	2.50	2.50	60.0	76	FAN, 2	20					
110	Chambal, Kota Bridge	NHAI	2018	kota, Rajasthan		650	3x50 + 350 + 3x50					Concrete											
111	Manair river bridge, Telangana	R & B Deppt, Govt of Telangaa	Ongoing	Karimanagar Telangana		500	140 + 220 + 140																
112	Kamla Park, Bhopal	Municipal Corporation of Bhopal	2017	Bhopal	3	220	55 + 110 + 55	15	15	2	Bearings	Composite	1.70	1.50	22.0	30.7	FAN, 2	11	0.24	0.19	0.018	NA	0.164
113	Mapusa river bridge	GSIDCL, Goa	2004	Goa		230	25 + 180 + 25	14	12	2	Bearings	Composite	1.50	1.50	40.0	60	FAN, 2	15					
114	Skywalk, Mumbai	MSRDC	2014	Mumbai		220	220	5	5	10	Bearings	Truss	4.50	4.50	50.0	60	Ellipse						
115	Thane Creek bridge	Thane Municipal Corporation	2012	Mumbai		230																	
116	Kharghar Junction, Navi Mumbai		2000	Navi Mumbai		60	2 x 30	28	24.2	2	Bearings	RCC deck	2.00	2.00	30.0	40	FAN, 2	12					
117	Mumbai - Pune Expressway	MSRDC	2000	Mumbai		44	2 x 22																
118	Kumbh Sthal on Mhanadi River, Raipur	PWD, Raipur	Ongoing	Raipur		300	75 + 150 + 75																
119	Wainganaga River bridge	PWD Nagpur	Ongoing	Nagpur	2 lane	690	70 + 4x140 + 70	15.26	7.5	2	Integral	concrete	2.00	1.20	30.0	5 & 8	fan, two	8 m					
120	Meethapur Junction, Patna	BRPNNL	2017	Patna	4 LANE	148	37 + 74.98 + 37	14.3	11	1	Bearings	Composite	2.00	2.00	14.0	14+10.8	Harp, Single	10.4					
121	Mahalakshmi Railway Station ROB	Municipal Corporation of Brihan, Mumbai	Ongoing	Mumbai	4 LANE	520	10 + 95 + 165 + 10x25	17.2	15	2	Integral	Composite	2.00	2.00	66.0	66+11	Fan Two	4/8					
122	Tapi River bridge	Surat Municipal Corporaion	2018	Surat, Gujarat	4 LANE	300	75 + 150 + 75	23.5	15	1	Integral	Concrete	3.00	3.00	34.875	34.875+15	Semi Harp, Single	7					
123	Kamatibaugh River bridge	Vadodara Municipal Corporation	2014	Vadodara, Gujarat	N.A.	82	30 + 52	4.75	4	1	Bearings	Composite	1.30	1.30	25.0	25+6	Fan, single	1.7 back stay & 9 fore stay					
124	Signature bridge, Delhi	DTTDC	2019	Delhi		395	251 + 4x36					Composite											
125	Canal Bridge	State Irrigation Deptt. 1982		Roorkee	Foot	73m	73m	1.6m		02	Bearings	Steel	1.20	1.20	10.0	10+5	Radiating	8					
126	Bridge over Ganges Channel	State Irrigation Deptt.	1988	Hardwar	2 LANE	130	2x65	10.5	7.5	02	Bearing	RCC	1.40	1.40	20.5	20.5+4.5	Radiating	8.13	0.75(M30)	0.043	0.016		0.25
127	Mumbai Pune Missing Link Bridge	MSRDC	Ongoing	Lonawala, Pune	6 Lane x 2 Decks	650	40 + 132.5 + 305 + 132.5 + 40	24.95	20.25	02 for each deck	Integral	Concrete - PSC	3.93	3.50	72.4	72.4+102.6	Harp Fan	10					
128	Ganga Bridge (Between Sultanganj and Aguwani Ghat)		India	road		1562.5 550	140 + 270 + 140	25.5		2			5.00	1.5	27.5	fan, 1 plane, fan, 2 plane	1.75						

NOTES: 1. While every attempt has been made to collect data from reliable sources, some information may not completely matc the actual structure
2. Blank spaces means the data could not be obtained

238

Annexure A2
(Pertaining to Chapter 7)

APPROACH TO DESIGN OF STAY CABLES AS PER INTERNATIONAL GUIDELINES

This Annexure presents philosophy and Methods for the design of cable for cable stayed bridges and extradosed bridges adopted by Japanese Code *"Specifications for Design and Construction of Cable Stayed Bridges and Extradosed Bridges"*, Setra – Stay Cable Recommendation and PTI Recommendations which are the codes available for design cable supported bridges. Also, recommendation of the FIB 89 are presented at the end.

I. Specifications for design and construction of cable stayed bridge and extradosed bridge - JPCI

As per Kasuga in his paper **[1]**, when designing structures that are reinforced using stays, rather than defining in advance whether the bridge will be a cable-stayed bridge or an extradosed bridge and then determining the allowable stress for the stays, the more rational approach would be to design the stays by focusing on the stress change caused by live loads that affect fatigue. This would make it possible to design each stay separately and enable the allowable stress to be set individually for each stay. From the outset, unlike suspension bridges, the stress change on a cable stayed bridge will differ depending on the stays, so it is not rational to define the allowable stress using a single value of 0.4 f_{pu}. This knowledge is reflected in JPCI specifications.

The specification allows two kinds of design method. One is normal fatigue design using fatigue load and design lifetime of a bridge (Method A). However, it is usually difficult to estimate the amount of future traffic and heavy trucks, especially in local roads. In that case, another approximation method using stay-cable stress change owing to designed vehicular live loads is introduced (Method B). **Fig. A2.1** shows the comparison of Methods A and B. As per Method B the stress variation in the cable due to design live load $\Delta\sigma_L$ is determined, based on $\Delta\sigma_L$, permissible stresses in cables at service loads is determined to carry our serviceability check. The above relation between the stress changes due to live and permissible stress is given for both prefabricated wire type and strand type cables.

Method B is defined such that it ensures adequate safety in comparison with bridges designed using Method A.

Fig. A2.1 shows fatigue stress range ($\Delta\sigma_{2E6}$) in the cable (Based fatigue design performed for 3 bridges). This fatigue stress range is compared to fatigue strength (f_{scrd}) divided by safety factor (γ_b).

For a strand stay cable fabricated on site using wedges, based on a system with fatigue strength of at least 120 N/mm^2 at 0.6 fpu or at least 200 N/mm^2 at 0.4f_{pu}. In this situation, γ_b is 1.4. The shaded section of the figure is the range determined by Method A ($\Delta\sigma_{2E6}$) with the fatigue design conditions indicated above. This is two-thirds of the $\Delta\sigma_L$, as prescribed by Method B, there is a safety factor of around 2.0 with respect to f_{scrd} /γ_b.

For a galvanised wire stay cable made at a factory as a cold-cast cable or as a cable with buttonhead anchorages, the relationship between the f_{scrd}/γ_b and the $\Delta\sigma_{2E6}$ estimation line is shown in **Fig. A2.1**, based on a system with fatigue strength of at least 180 N/mm^2 at 0.6 fpu or at least 230 N/mm^2 at 0.4 fpu, and like the figure for the cable fabricated on site. It can be seen from the figure that the factory-made cable also has a safety factor of around 2.0 with respect to f_{scrd}/γ_b.

Therefore, in the stays designed by Method B, Δ_{sL} is determined to require a safety factor of about 2.0 for $\Delta\sigma_{2E6}$ with respect to f_{scrd}/γ_b.

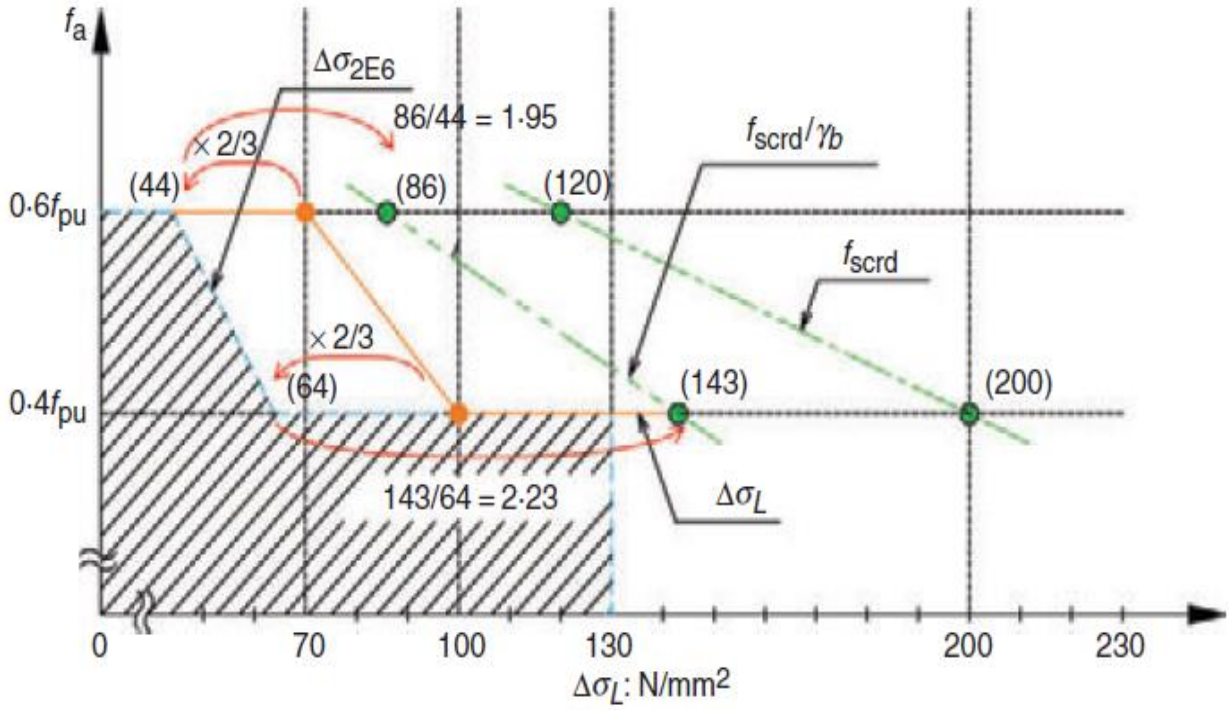


Fig. A2.1 : Comparison of Method A and Method B

According to this specification designer can choose the tensile stress in each stay cable from $0.4f_{pu}$ to $0.6f_{pu}$ **Fig. A2.2** continuously and not one value of tensile stress in one bridge.

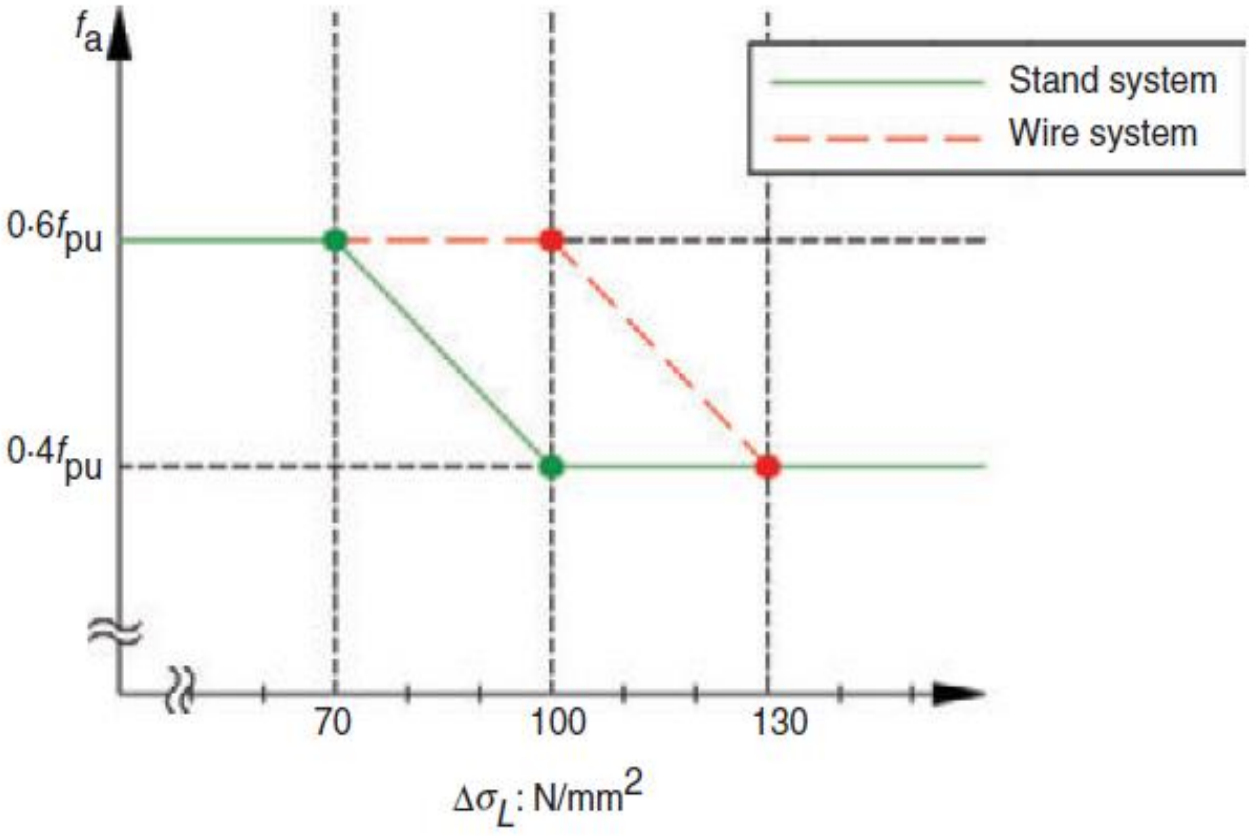


Fig. A2.2 Allowable stress in SLS to Design LL stress variation

Fig. A2.3 shows the design flow for stay-cable design.

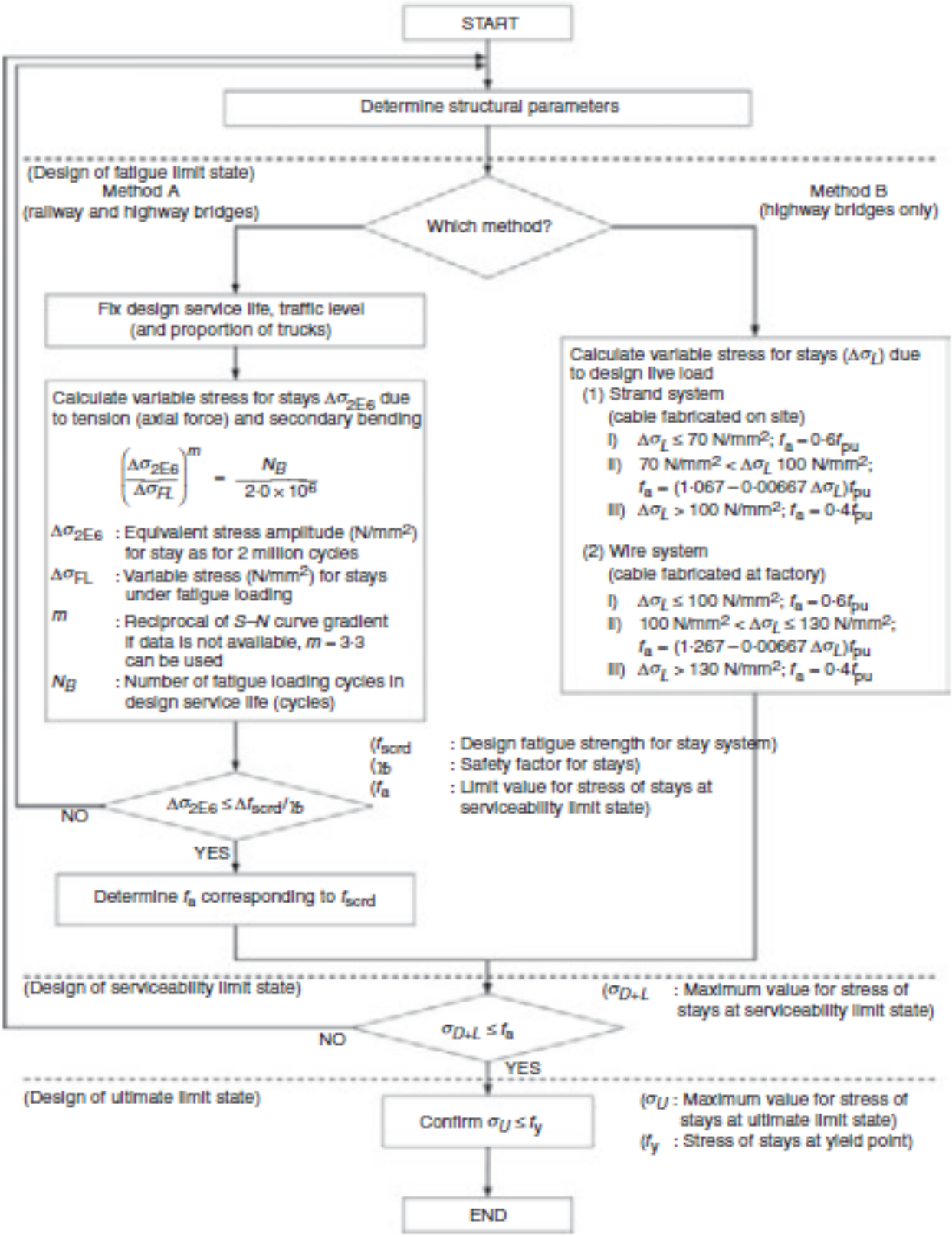


Fig. A2.3 Flow Chart for Stay Cable design as per JPCI Specifications

II. Setra Recommendation - Cable Stays

The SETRA recommendations relate the allowable axial stress σa at SLS for the stay cables with the stress variation ΔσL due to SLS live loads and can be determined from the following equation. The allowable axial stress σ_a is applicable for the entire stay cable system [SETRA 2002] unlike JPCI recommendation which applies to individual cables.

$$\sigma_a = \begin{cases} 0.6\,\sigma_{UTS} & \text{if } \Delta\sigma_L \le 50\,\text{MPa} \\ \sigma_a \le 0.46\left(\dfrac{\Delta\sigma_L}{140}\right)^{-0.25}\sigma_{UTS} & \text{if } 50 < \Delta\sigma_L \le 140\ \text{MPa} \\ 0.46\,\sigma_{UTS} & \text{if } \Delta\sigma_L > 140\,\text{MPa} \end{cases}$$

The SLS design method demonstrated above involves implicitly the fatigue consideration for the stay cable system.

The explicit fatigue limit state design requires the stay cable system to be tested for two million cycles for specific upper stress limits and specific amplitudes. For cable stayed bridges, the upper stress limit is taken usually as 45% f_{UTS}**.** The amplitude is commonly taken as 160 MPa for strand systems and 200 MPa for wire systems. Upon carrying out the fatigue test, appropriate safety factors is taken into account to specify the allowable stress variation.

R. Walter [Walter et al. 1999] suggests simplified method for the definition of the permissible stress variation $\Delta\sigma_{per}$ by establishing the Woehler curve **Fig. A2.4** from the available test results then deducing the allowable stress variation $\Delta\sigma_{per}$ by using the following equation:

$$\Delta\sigma_{per} = \frac{\Delta\sigma_{test}}{\gamma_{Mf}} \approx \frac{\Delta\sigma_{test}}{1.5}$$

Where:

γ_{Mf} =1.5 partial safety factor (as per SETRA)

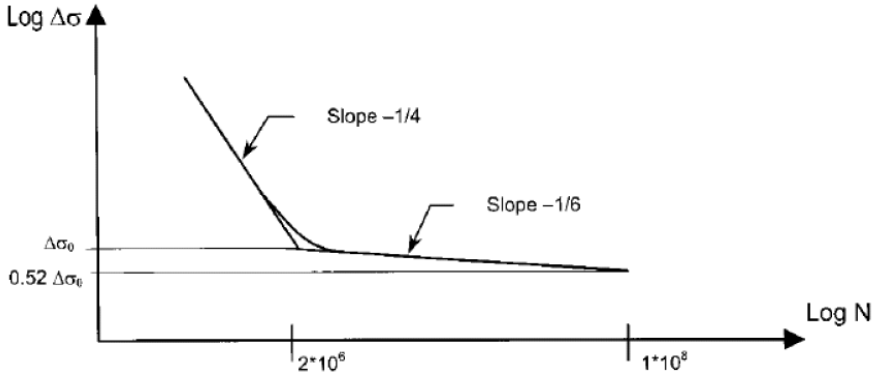


Fig. A2.4 Typical fatigue Curve for Stay Cable as per SETRA 2002

Upon the determination of $\Delta\sigma_{test}$, the stress variation due to the fatigue load should be calculated (using a structural computer model) and should be less than (0.52 $\Delta\sigma_0$ /1.5). For a stay cable system with $\Delta\sigma_0$ = 200 MPa for example, the allowable stress variation due to the fatigue load = (0.52 x 200/1.5) ≈ 70 MPa. The fatigue load for road bridges for design shall be as per the Eurocode EN 1991-2-2003. The stress due to fatigue live load should be less than 70 MPa as per SETRA.

Based on the previous experience with the design of cable-stayed bridges with concrete decks, the stress variations due to fatigue loads like that of the Model 3 of Eurocode EN 1991-2-2003, are often far below 70 MPa. The SLS design governs therefore the required size of the stay cables. Similarly, for extradosed bridges, where the stress variations due to live loads are less than the corresponding stress variations for the cable-stayed bridges, the SLS design is expected to also govern the size of the stay cables and the permissible stress at SLS is increased as allowed for by SETRA recommendations (2002).

III. PTI Recommendations for Stay cable design, Testing and Installation

The philosophy adopted by PTI for performance of the cable, is to start with the single 7-wire element fatigue capacity of 33 ksi (229 MPa) **Fig. A2.5**, then reduce this by a value, Δ_1, to cover quality assurance considerations, and a value, Δ_2, to cover length effects, anchorage stress risers and factor of safety. The quality assurance value considers the difference from the performance of an individual strand element compared to a group of strands forming a cable. It includes interaction effects, cable. twist, and assembly variances, and is based on experience from load testing results of many assembled cables. For strand cables the Δ_2 correction used by PTI is 10 ksi (69 MPa) and the Δ_1 correction is 4.93 ksi (35 MPa). **Fig. A2.5** shows application of these corrections, resulting in an allowable fatigue stress range for a cable comprised of multiple 7-wire strands of 19 ksi (124 MPa) at 2 million cycles, based on a maximum stress of 45% of the ultimate strength of the strand

The PTI 3rd edition and earlier were based on an allowable stress design procedure where the cables were sized based on a maximum stress of 0.45 f’s for Group I loading, then a check of the fatigue range of 18.1 ksi based on the selected cable size. If based on this check the fatigue limit is not met, then fatigue governs, and the cable size is increased to reduce the stress range.

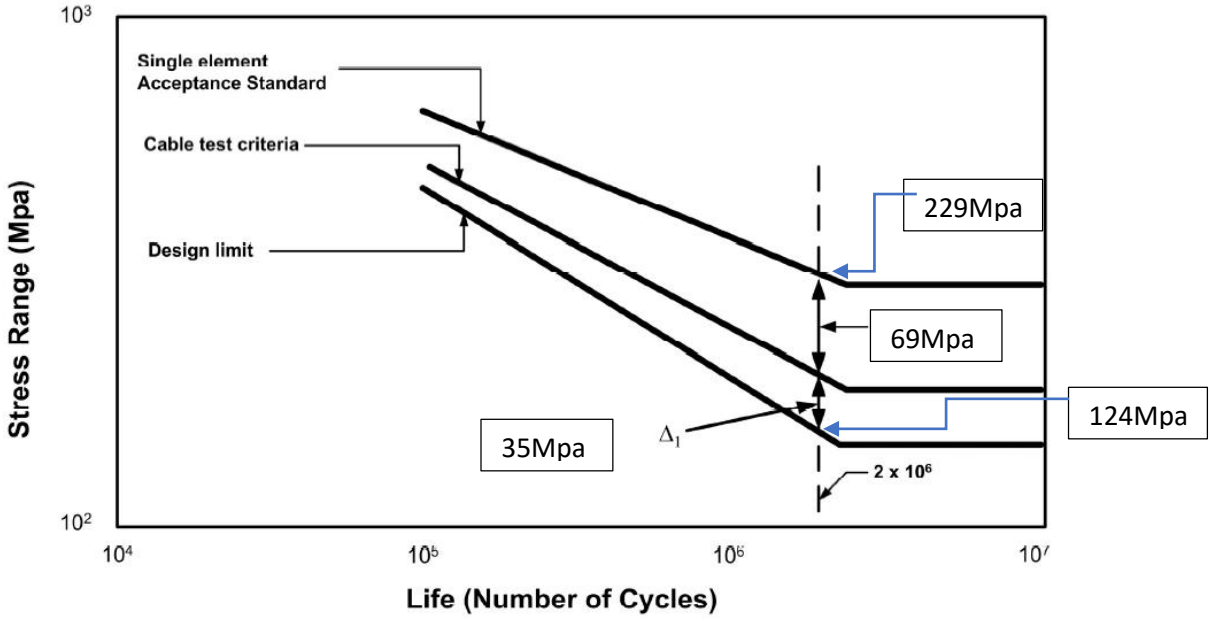


Fig. A2.5 Strength resistance Factor, (PTI DC45.1-18)

The current PTI specification is based on a Load Resistance Factor Design (LRFD) philosophy, meaning that the four basic limit states of service, strength, fatigue and extreme event must satisfy for the following expression:

$$\eta \sum \gamma_1 Q_i \leq \phi R_n = R_r$$

where :

γ_1 is the load factor

ϕ is the resistance factor

Q_i is the force effect, axial load, or axial plus bending, as applicable

R_n is the nominal resistance.

R_r is the factored resistance; and

$\eta >$ 0.95 is a factor relating to ductility, redundancy, and operational importance. (Refer to Sections 1.3.3, 1.3.4, and 1.3.5 of AASHTO LRFD.). The factor η = 1 for these Recommendations.

For the limit states referred to in Table 3.4.1-1 of AASHTO LRFD, the following resistance factors shall apply:

Strength A – Axial Only ϕ = 0.65 to 0.75 per diagram in **Fig. A2.6**

Strength B – Axial and Bending combined $\phi = 0.78$

Extreme Event $\phi = 0.95$

Fatigue $\phi = 1.0$

In PTI recommendations", 5th Edition an attempt to include term extradosed bridges in the specification was made. These provisions allowed design to an equivalent allowable stay stress of 0.6 f's for the very specific class of bridges where the fatigue stress range is less than 2.5%. While this went part way in recognizing extradosed bridges, it allowed no flexibility on intermediate conditions where the fatigue stress is more than 2.5% but less than a typical cable-stayed bridge.

PTI, 6th Ed. edition removed the specific references to extradosed bridges in the 5th edition, to include provisions that address bridges with relatively low live load demands on the stay cables in a more general manner. This is accomplished by allowing a transition in the stay capacity for low demand cables, expressing this as a function of the total live load plus wind stress to maximum ultimate tensile strength ratio (Total LL+W/MUTS) for the cable. The material factor are considered to recognize the lower live load fatigue demand on bridges that have relatively small live load demand on the cables. For the total LL+W/MUTS ratio over 7.5% the material factor is 0.65 as for a normal cable stayed bridge. For a Total LL+W/MUTS ratio of 2.5% a factor of 0.75 is allowed, essentially allowing a maximum stay stress of 0.6 f's. A linear transition is permitted between these two limits **Fig. A2.6**.

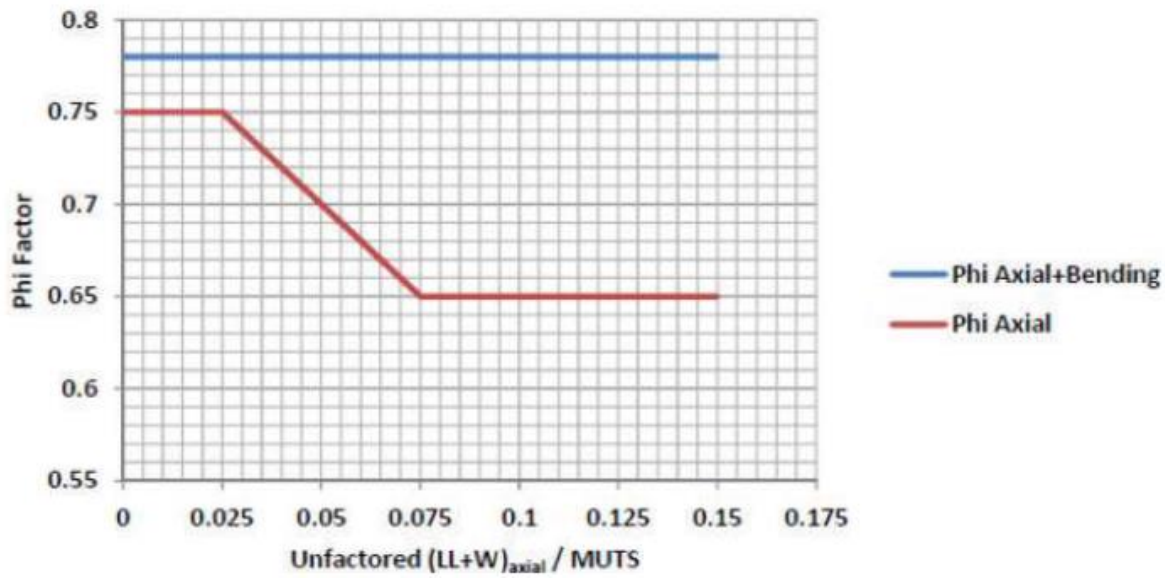


Fig. A2.6 Strength resistance Factor, (PTI DC45.1-18)

The PTI Recommendations requires that the cable forces comply with a Strength Limit State and a Fatigue Limit State. For a theoretically infinite fatigue life, the maximum stress range in each cable must be less than the constant amplitude fatigue threshold stress. The fatigue load consists of a single design truck, in a single lane, and the load effect is then increased by a Dynamic Load Allowance of 15% and by a factor of 1.4 to account for longer spans of cable-stayed bridges than those for which the fatigue provisions of the AASHTO

LRFD (AASHTO 2004) were originally designed. The factored load effect γ(Δ*F*) must be less than half the constant amplitude fatigue threshold $(\Delta F)_{TH}$ as described by the following equations.

$$\gamma(\Delta F) \leq (\Delta F)_{\eta}$$

$$\gamma(\Delta F) \leq \frac{1}{2}(\Delta F)_{TH}$$

where (Δ*F*) is the stress range due to the passage of the fatigue load

$(\Delta F)_{TH}$ is the constant amplitude fatigue threshold (taken as 110 MPa for parallel strands)

γ is the load factor of 0.75

$\therefore$ 0.75 (Δ*F*) $\leq \frac{1}{2}$ (110)

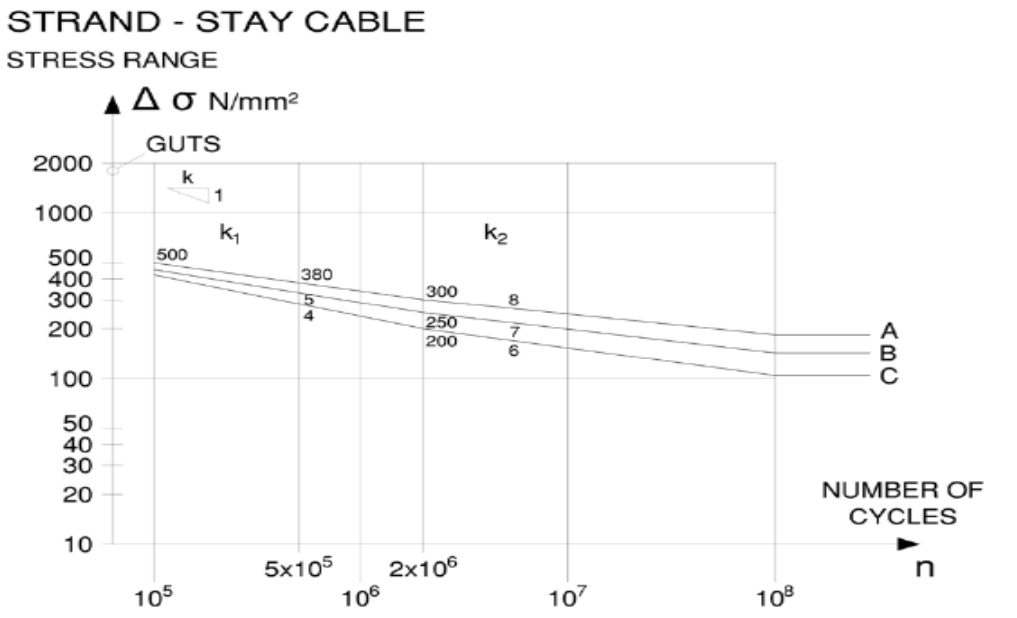
(Δ*F*) ≤ 73 MPa

For AASHTO LRFD, stress range due to fatigue live load must be less than 73 MPa.

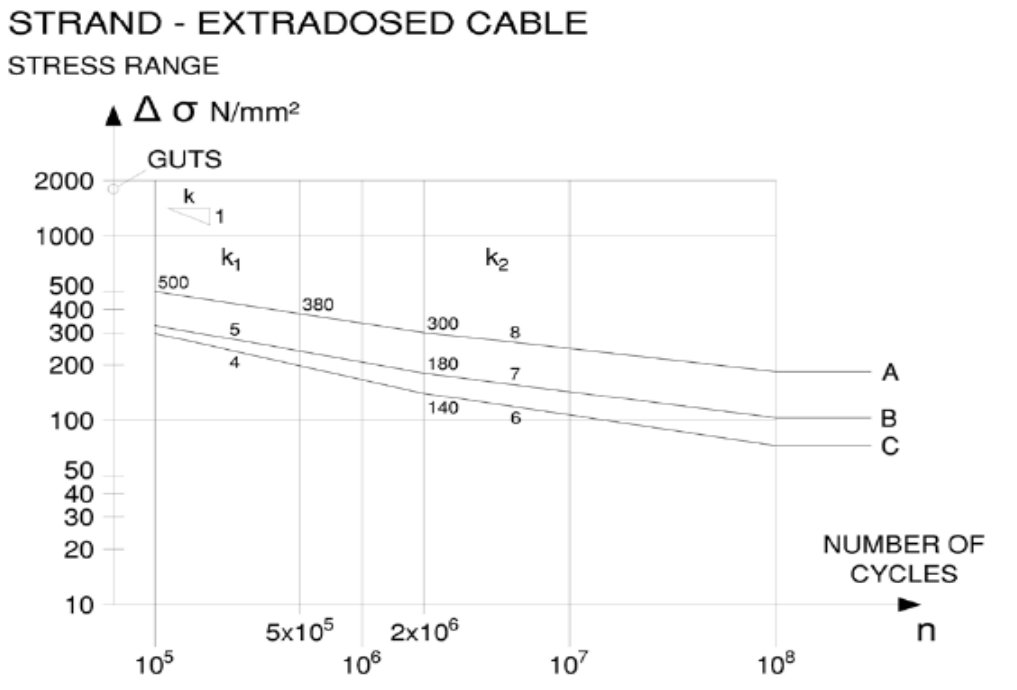
IV. Acceptance of Cable systems using prestressing steel – FIB 89

As per FIB 89, fatigue design should be carried out for stay cables by the method similar to SETRA as explained above with material factor of 1.25 and Wohler Curve C **Fig. A2.7** which represents minimum test performance requirements of stayed / Extradosed cables.

Also the permissible stresses in the stay cables system tested in accordance with FIB 89 **Chapter 6** shall be 50% GUTS for cable stay and for extradosed cable system shall be 60% GUTS.



A = SINGLE TENSILE ELEMENT, UPPER STRESS 45% AUTS
B = ANCHORED SINGLE TENSILE ELEMENT, UPPER STRESS 45% GUTS
C = CABLE TEST, UPPER STRESS 45% GUTS



A = SINGLE TENSILE ELEMENT, UPPER STRESS 45% AUTS
B = ANCHORED SINGLE TENSILE ELEMENT, UPPER STRESS 55% GUTS
C = CABLE TEST, UPPER STRESS 55% GUTS

Fig. A2.7 S-N Curve for cable system as per FIB 89

Annexure A3
(Pertaining to Chapter 9)

Additional Information on Wind Tunnel Testing

This Annexure provides more information about wind tunnel testing of bridge using (i) section model (ii) Full bridge model

Tests on section models of bridge decks can be used to determine the mean or static components of the overall wind load on the model. These wind loads can be obtained using rigid models with geometrically scaled features. The effect of wind inclination in elevation should be examined. The extent of the effect should be judged depending on the site topography, any planned super-elevation of the bridge and predicted torsional deflections under traffic loads. Generally, tests up to ± 6° are adequate. Accurate measurements of both the mean and the dynamic components of the overall loads can only be obtained if both the approach flow and the local environment are properly simulated. For the scale of model bridge required, this becomes impractical.

The other important objective of section model tests is to determine the aerodynamic stability of the bridge deck, mounted with deck furniture, using a geometrically scaled model of a section of the bridge elastically mounted in a wind tunnel. Typically, such models simulate the lowest bending and torsional vibration frequencies, and are tested in uniform laminar flow. This is expected to yield conservative results, as compared to those obtained in turbulent flow. In more advanced or refined stages, section models are tested in simulated turbulent flow in order to provide estimates of the responses at sub-critical wind speeds. As the simulated turbulence generally has a preponderance of the smaller-size eddies most likely to influence flow features such as vortex-shedding or re-attachment, the total intensity of turbulence should be selected with care. Generally, this should be significantly lower than the standard atmospheric value for full scale. Reliance on beneficial effects from turbulence should not be allowed to reduce the likely aerodynamic effects. In addition to modelling the geometry, it is necessary to maintain a correct scaling of inertia forces, the time scale, the frequency, and the structural damping. The time scale is normally set indirectly by maintaining the equality of the model and full-scale reduced velocities of particular modes of vibration. The reduced velocity is the ratio of a reference wind speed and the product of a characteristic length and the relevant frequency of vibration.

Measurements should be carried out through the range of wind speeds likely to occur at the site to provide information on both relatively common events, influencing serviceability, and relatively rare events, which govern ultimate strength behaviour. Wind inclination in elevation should be examined.

Measurements of vortex excitation require careful control of the wind speed around the critical velocity, and any predicted divergent amplitudes should be assessed to ensure they do not become so violent as to destroy the model.

A Sectional model test can furnish most of the relevant information required for the aerodynamic design of the bridge. One limitation in modelling herein is that the Reynolds number can not be simulated. However, fortunately this distortion has insignificant effect because of the deck section having sharp edges. Furthermore, it is to be remembered that details at the edge of the section can have a considerable effect on its aerodynamic response. Thus, attachments such as, fairings, guide vanes and deflectors are considered for this purpose.

A dynamic model of the full bridge, commonly referred to as an aero-elastic model, can be used in the wind tunnel, to provide information on the overall wind induced mean and/or dynamic loads and responses of bridges. Such models are particularly valuable for slender, flexible and dynamically sensitive structures, where dynamic response effects may be significant. To be representative the tests should consistently model the salient characteristics of natural wind at the site and the aerodynamically significant features of the geometry of the bridge. It is also necessary to correctly model the stiffness, mass and damping properties of the structural system.

It is only possible to model the full spectrum of atmospheric turbulence in a wind tunnel at small scale; together with the obvious constraint of fitting a full bridge model within the tunnel, this is generally irreconcilable with the scale desirable to ensure correct behaviour, which is commonly sensitive to small changes in cross-section. A full model should therefore be used only where non-uniformity of section or of incident flow conditions, complex dynamics or erection considerations necessitate its use. Particular care is needed in its design and interpretation of the test results. It is noteworthy that modern techniques of model fabrication and very highly sophisticated instruments such as accelerometers, anemometers, and pressure transducers/scanners make it possible to monitor model responses very accurately.

ANNEXURE A4
(Pertaining to Chapter 11)

A4.1. APPROACHES OF INTERNATIONAL GUIDELINES FOR TESTING OF STAY CABLE

Table A4.1

INTERNATIONAL CODES	SETRA [1]	fib-89 (CEB-FIP) [2]	PTI (7th EDITION) [3]
1 CABLE STAY BRIDGES	✓	✓	✓
2 EXTRADOSED BRIDGES	✓	✓	
UPDATED	First print published in 2002 & the same will not be updated further.	Latest Version published in March- 2019	Latest Version published in Nov- 2018

Notes:-

1 If for a particular project, all the cables have $\Delta\sigma_{sls}$ (stress range in cables due to vehicular live load under frequent live load in MPa) less than 50 MPa than, qualification testing (initial as well as suitability testing) requirements of the extradosed bridges shall be adopted else all the testing requirements of the cable stayed bridges shall be adopted.

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

A4.2. COMPARISON OF INTERNATIONAL CODES FOR STAY CABLE FOR CABLE STAYED BRIDGES & RECOMMENDED CODE TO BE FOLLOWED FOR TEST

TESTING OF CABLE STAY- ANCHORAGE ASSEMBLY ✓

OR

TESTING OF CABLE STAY- SADDLE ASSEMBLY ✓

Table A4.2

	Test Detail	SETRA [1]	fib-89 (CEB-FIP) [2]	PTI (7th EDITION) [3]	Recommended Code For Testing
S.No	**INITIAL TESTING**				
1	ANCHORAGE /SADDLE AXIAL FATIGUE & TENSILE TESTING	✓✓	✓✓	✓✓	**fib-89**
2	ANCHORAGE /SADDLE BENDING FATIGUE & TENSILE TESTING		✓✓		**fib-89**
3	ANCHORAGE / SADDLE LEAK TIGHTNESS TESTING	✓✓	✓✓	✓✓	**fib-89**
4	SADDLE FRICTION TESTING	✓	✓	✓	**fib-89**
5	FIRE RESISTANCE QUALIFICATION TESTING (OPTIONAL-SPECIFIC TO PROJECT REQUIREMENT)-Given in PTI only			✓	**PTI**
	SUITABILITY TESTING				
1	SINGLE ELEMENTS (STRAND ALONE)-FATIGUE TESTING	✓	✓	✓	**fib-89**
2	ANCHORAGE AXIAL FATIGUE & TENSILE TESTING	✓✓	✓✓	✓✓	**fib-89**
3	ANCHORAGE BENDING FATIGUE & TENSILE TESTING	NOT REQUIRED TO BE PERFORMED			
4	ANCHORAGE LEAK TIGHTNESS TESTING	NOT REQUIRED TO BE PERFORMED			
5	SADDLE FRICTION TESTING	✓	✓	✓	**fib-89**
6	FIRE RESISTANCE QUALIFICATION TESTING (OPTIONAL-SPECIFIC TO PROJECT REQUIREMENT)-Given in PTI only			✓	**PTI**

A4.3 COMPARISON OF INTERNATIONAL CODES FOR STAY CABLE TESTING FOR EXTRADOSED BRIDGES & RECOMMENDED CODE TO BE FOLLOWED FOR TEST

TESTING OF EXTRADOSED STAY- ANCHORAGE ASSEMBLY ✓

OR

TESTING OF EXTRADOSED STAY- SADDLE ASSEMBLY ✓

Table A4.3

	Test Detail	SETRA [1]	fib-89 (CEB-FIP) [2]	PTI (7th EDITION) [3]	Recommended Code For Testing
S.No	**INITIAL TESTING**				
1	ANCHORAGE /SADDLE AXIAL FATIGUE & TENSILE TESTING	✓✓	✓✓		**fib-89**
2	ANCHORAGE / SADDLE LEAK TIGHTNESS TESTING	✓✓	✓✓		**fib-89**
3	SADDLE FRICTION TESTING	✓	✓		**fib-89**
4	FIRE RESISTANCE QUALIFICATION TESTING (OPTIONAL-SPECIFIC TO PROJECT REQUIREMENT)-Given in PTI only			✓	**PTI**
	SUITABILITY TESTING				
1	SINGLE ELEMENTS (STRAND ALONE)-FATIGUE TESTING	✓	✓		**fib-89**
2	ANCHORAGE AXIAL FATIGUE & TENSILE TESTING	✓✓	✓✓		**fib-89**
3	ANCHORAGE LEAK TIGHTNESS TESTING	NOT REQUIRED TO BE PERFORMED			
4	SADDLE FRICTION TESTING	✓	✓		**fib-89**
5	FIRE RESISTANCE QUALIFICATION TESTING (OPTIONAL-SPECIFIC TO PROJECT REQUIREMENT)-Given in PTI only			✓	**PTI**

A4.4. Applicable Property, Acceptance Criteria & Test Method & Frequency of Testing of Materials

Table A4.4.1 Recommended Values for Prestressing Steels for Cables

Type of steel	E-modulus	Diameter	Tensile strength	Cross section	Recommended tolerances on mass	Characteristic ultimate force
	GPa	mm	MPa	mm^2	%	kN
Strands	195 ± 5%	15.2	1860	140	± 2	260.4
		15.7	1770	150	± 2	265.5
			1860			279.0
Wires	205 ± 5%	6	1770	28.27	± 2	50.1
		7	1670	38.5		64.3
		7	1770	38.5	± 2	68.1

Table A4.4.2 Mechanical Characteristics for Prestressing Steels

Property			Acceptance Criteria	Test Method
Elongation at maximum force			Not less than 3.5%	ISO 15630-3[S7]
Constriction at break			Ductile break visible to naked eye, constriction coefficient ≥25%	ISO 15630-3
Relaxation 1000h at 0.70 AUTS, 20^0C			Not more than 2.5%[1)]	ISO 15630-3
Fatigue stress range with an upper stress limit of 0.45 AUTS	Strands	300 MPa	At least $2x10^6$ cycles	ISO 15630-3
	Wires	370 MPa		
Fatigue stress range with an upper stress limit of 0.45 AUTS (alternative to 2×10^6 cycles)	Strands	380 MPa	At least 5×10^5 cycles	ISO 15630-3
	Wires	465 MPa		
Fatigue stress range with an upper stress limit of 0.45 AUTS (alternative to 5×10^5 cycles)	Strands	500 MPa	At least 1×10^5 cycles	ISO 15630-3
	Wires	610 MPa		
Tensile test after the fatigue test			95% of AUTS	ISO 15630-3
Deflected tensile test of Strands			Not more than 20%	ISO 15630-3

[1)]The values are applicable for testing of samples taken just after manufacturing.

Above specified properties apply to hot dipped metallically coated or epoxy coated prestressing steel. However, they also have to be satisfied after application of the filling material and sheathing.

Table A4.4.3 Requirements for metallic coatings on prestressing steel[S12]

Property		Acceptance Criteria	Test Method
Zinc or zinc/aluminium coating	-Continuity	No defects	EN 10244-1 and 2[S13]
	-Weight of coating	190 to 350 g/m^2	
	-Adherence	No defects, cracks	
	-Appearance	Smooth, no zinc drop	

Table A4.4.4 Requirements for epoxy coatings on prestressing steel

Property	Acceptance Criteria	Test Method
Coating thickness	380 to 1140 µm	ASTM A 882/A882M[S14]
Salt spray test	No corrosion, holidays[1)] nor other coating damage present after 3,000 hours exposure to salt spray (fog)@ 70% GUTS	ASTM B117[S4]
Chloride permeability	No chloride penetration detected	FHWA-RD-74-18
Chemical resistance	2 types of samples tested: Normal epoxy coated strand and sample with 1/4" hole intentionally drilled into coating. No coating damage after 45 days immersion. Steel corrosion at drilled hole did not cause adjacent epoxy coating to soften, blister, nor lose bond with adjacent steel.	ASTM G 20[S49]
Impact test	No cracking, shattering or bond loss	ASTM G 14[S23]
Sand abrasion test	< 10 mils (0.25 mm) loss of coating	ASTM D 968[S50]
Bending test	No cracking or debonding when bending strand around mandrel 32 times nominal strand diameter	ASTM A 370[S51]
Adhesion of coating	No coating peel-off is found after the break in the tensile test except in the area near breaking position and chucking position	ASTM A 882/A882M[2)]

[1)]Holiday detection: Dry-Type continuous in line detector with minimum voltage of 3KV would be applied.

[2)]No cracks visible to the unaided eye shall occur in the coating upto an elongation of 1% , or no coating peel-off is found after the break in the tensile test except in the area near breaking position and chucking position.

Table A4.4.5 Grease Specification

Property		Acceptance Criteria	Test Method
Cone penetration,60 strokes at 25^0 C (1/10 mm)		220 – 300	ISO 2137[S24] or ASTM D217[S25]
Dropping point		≥ 150^0 C	ISO 2176[S26] or ASTM D566[S27]
Heat resistance, flash point		≥ 250^0 C	EN ISO 2592[S40]
Oil separation	At 40^0C	At 72 hours : ≤ 2.5% At 7 days : ≤ 4.5%	BS 2000-121[S28] DIN 51 817[S29] NFT 60 – 191[S30] or ASTM D 6184[S31]
	At 100^0C	At 50 hours : ≤ 4%	
Oxidation stability		100 hours at 100^0C: ≤ 0.06 MPa 1000 hours at 100^0C: ≤ 0.2 MPa	DIN 51 808[S32] Or ASTM D942[S33]
Corrosion Test[1)]	168 hours at 35^0C	No corrosion	EN ISO 9227 (NSS test)[1),S34]
	168 hours at 35^0C	No corrosion	EN ISO 6270-2 (AHT test)[S35] Or ISO 9227 (distilled water instead of NSS)
Rust prevention		Grade : 0	ISO 11007[S36] , solution of sodium chloride
Content of aggressive elements	Cl^-,S^{2-}, NO^-_3	≤ 50 ppm (0.005%)	NFM 07-023[S37,2)]
	SO_4^{2-}	≤ 100 ppm (0.010%)	NFM 07-023[2)]

[1)]Test sample consists of a structural steel plate S355 with a surface roughness comparable to prestressing wire and strand. The plate is covered with a layer of grease of a maximum thickness of 125 µm. This thickness can be controlled with eddy current or by measuring the mass.

[2)]Applied accordingly to grease. Requirements for grease pertaining to fluidity, de-oiling , water absorption, saponification, micro-biological resistance, coefficient of expansion must be satisfied or declared.

Table A4.4.6 Wax Specification

Property	Acceptance Criteria	Test Method
Congealing point	≥ 65^0C	ISO 2207[S38]
Heat resistance, dropping point	≥ 60^0C	ISO 2176[S39]
Heat resistance, flash point	≥ 250^0C	ISO 2592[S40]
Penetration (1/10 mm) at 25^0C	≤ 125 (in 1/10 mm)	ISO 2137[S41]
Cold resistance	No cracking at -40^0C	ISO 2137[S41]

Bleeding at 40^0C		≤1.0%	BS 2000-121[S28] DIN 51 817[S29] NFT 60 – 191[S30] or ASTM D 6184[S31]
Resistance to oxidation 100 hours at 100^0C		≤ 0.03 MPa	ASTM D942[S33]
Copper-strip corrosion 100 hours at 100^0C		Class 1a	ISO 2160[S42]
Corrosion Test[1)]	168 hours at 35^0C	No corrosion	EN ISO 6270-2 (AHT test)[S35] Or ISO 9227 (distilled water instead of NSS)[S34]
	168 hours at 35^0C	No corrosion	ISO 11007[S36] , solution of sodium chloride
Content of aggressive elements	Cl^-,S^{2-}, NO^-_3:	≤ 50 ppm (0.005%)	NFM 07-023[S37]
	SO_4^{2-} :	≤ 100 ppm (0.010%)	NFM 07-023

[1)]Test sample consists of a structural steel plate Fe 510 with a surface roughness comparable to prestressing wire and strand. The plate is covered with a layer of wax of a maximum thickness of 125 µm. This thickness can be controlled with eddy current or by measuring the mass.

Table A4.4.7 Characteristics of PE sheathing

Property	**Acceptance Criteria**	**Test Method**
Melt index	To be declared	ISO 1133[S16]
Specific weight, Density	≥ 0.94 g/cm^3	ISO 1183[S17]
Carbon black	2.3 ± 0.3%	ISO 6964[S18]
Dispersion of the carbon black	Index is max. 3	ISO 18553[S19]
Distribution of the carbon black	Index is max. C 2	ISO 18553
Tensile strength 50 mm/minute (test speed)	≥ 22 MPa on raw material ≥ 18 MPa on sheathing	ISO 527-2[S20]
Elongation at break at 23^0 C 50 mm/minute (test speed)	≥ 600% on raw material ≥ 250% on sheathing	ISO 527-2
Elongation at break at -20^0 C 50 mm/minute (test speed)	≥ 150% on raw material ≥ 100% on sheathing	ISO 527-2
Thermal stability under O_2	≥ 30 minutes at 210 ^{0}C, without degradation	ISO11357-6[S20]

Table A4.4.8 Characteristics of the sheathed cable strands

Property	Acceptance Criteria	Test Method
Sheathing thickness	≥ 1.5 mm	Calibrated gauge
Quantity of filling material	≥ ± 20% of specified value	Calibrated gauge
Friction resistance (specimen length of 0.3)	≥ 667 N	XP A 35-037.1[S5]
Leak tightness / migration[1)]	Increase mass due to absorption Of water shall be less than 0.8 g.	XP A 35-037.1 Static test
Impact resistance	No perforation of the sheathing	XP A 35-037.1, 3 ASTM G14[S23]

[1)]The value are applicable for testing of samples taken just after manufacturing.

Table A4.4.9 Characteristics of HDPE cable pipe

Property	Acceptance Criteria	Test Method
Melt index	To be declared	ISO 1133[S16]
Specific weight, Density	≥ 0.94 g/cm^3	ISO 1183[S17]
Carbon black[1)]	2.3 ± 0.3%	ISO 6964[S18]
Dispersion of the carbon black[1)]	Index is max. 3	ISO 18553[S19]
Distribution of the carbon black[1)]	Index is max. C 2w	ISO 18553
Tensile strength 50 mm/minute (test speed)	≥ 22 MPa on raw material ≥ 18 MPa on pipe	ISO 527-2[S20]
Elongation at break at 23^0 C 50 mm/minute (test speed)	≥ 600% on raw material ≥ 350% on pipe	ISO 527-2 ISO 6259-3[S46]
Elongation at break at -20^0 C 50 mm/minute (test speed)	≥ 150% on raw material ≥ 100% on pipe	ISO 527-2, ISO 6259-3
Thermal stability under O_2	≥ 30 minutes at 210 ^{0}C, without degradation	ISO 11357-6[S20]
Thermal coefficient of dilation	Value to be declared by manufacturer	ISO11359-2[S22]
Bending modulus	≥ 750 MPa at 23 ^{0}C on raw material ≥ 600 MPa at 23 ^{0}C on pipe	ISO 178[S45]
Environmental stress cracking resistance	Test with min of 600 hours Cond C F20	ISO 22088[S55] or ASTM D1693[S44]

A4.5. Quality control testing of cable systems

Property	Test Frequency
Prestressing steel according to Table A4.4.2 Mechanical characteristics for prestressing steels	
Tensile strength with elongation at maximum force	1 test per strand coil[1)]
Constriction at break	
Relaxation	1 test par batch of prestressing steel[2)]
Fatigue strength with subsequent tensile test	2 tests every 100 tonnes of strand[3)]
Deflected tensile test	1 test every 100 tonnes of strand[3)]
Metallic coating according to Table A4.4.3 Requirements for metallic coating on prestressing steel[S12]	
Continuity	1 test every 20 tonnes of strand[3)]
Weight of coating	
Adherence	
Epoxy coating according to Table A4.4.4 Requirements for epoxy coating on prestressing steel	
Coating thickness	1 test per strand coil[1)]
Salt spray test	Only for initial type testing, not applicable for factory production control
Chloride permeability	
Chemical resistance	
Impact test	
Sand abrasion test	
Bending Test	1 test per strand coil[1)]
Adhesion of coating	1 test per strand coil[1)]
Relaxation	1 test per batch of prestressing steel[2)]
Grease filling material according to Table A4.4.5 Grease specification	
Cone penetration	1 test per batch[5)] of grease
Dropping point	
Heat resistance, flash point	
Oil separation at 40° C	
Oil separation at 100° C	
Oxidation stability	Only for initial type testing, not applicable for factory production control
Corrosion test (NSS test)	
Corrosion test (AHT test)	

Rust prevention	1 test per batch[5)] of grease
Content of aggressive elements	
Wax filling material according to Table A4.4.6 Wax specification	
Congealing point	1 test per batch[5)] of wax
Heat resistance, dropping point	
Heat resistance flash point	
Penetration at 25° C	
Cold resistance	
Bleeding at 40° C	
Resistance to oxidation 100h at 100° C	Only for initial type testing, not applicable for factory production control
Copper-strip corrosion 100h at 1 00° C	
Corrosion test (NSS test)	
Corrosion test (AHT test)	
Content of aggressive elements	1 test per batch[5)] of wax
Sheathing according to Table A4.4.7 Characteristics of PE sheathing	
Melt index	1 test per batch of raw material[4)]
Specific weight, Density	
Carbon black	
Dispersion of carbon black	
Distribution of carbon black	
Tensile strength	1 test per batch of raw material[4)] 3 tests every 10 tonnes of sheathing material
Elongation at break at 23^0 C	
Elongation at break at -20^0 C	
Thermal stability under O^2	1 test per batch of raw material[4)]
Completed strand according to Table A4.4.8 Characteristics of the sheathed cable strands	
Sheathing thickness	1 test per strand coil[1)]
Quantity of filling material	1 test per strand coil[1)]
Friction resistance	3 tests every 100 tonnes of strand[3)]
Leak tightness	
Impact resistance	
HDPE cable pipe according to Table A4.4.9 Characteristics of HDPE cable pipe	
Melt index	1 test per batch of raw material (on raw material[4)] 1 test per size of cable pipe but at least one test

	per raw material batch (on pipe)
Specific weight, Density	1 test per batch of raw material[4)]
Carbon black	
Dispersion of carbon black	
Distribution of carbon black	
Tensile strength	3 tests per batch of raw material (on raw material[4)] 3 test per size of cable pipe in the project (on pipe)
Elongation at break at 23^0 C	
Elongation at break at -20^0 C	
Thermal stability under O^2	1 test per size of cable pipe but at least one test per raw material batch (on pipe)
Thermal coefficient of dilatation	Only for initial type testing, not applicable for factory production control
Bending modulus	3 tests per size of cable pipe but at least one test per raw material batch (on pipe)
Environmental Stress Cracking Resistance	Only for initial type testing, not applicable for factory production control

[1)]A coil is a delivery unit, weighing not more than 5 tonnes.

[2)]Batch corresponding to one heat number, weighing not more than 200 tonnes. Relaxation tests with minimum 120 h according to ISO 15630-3 and subsequent extrapolated relaxation values to 1000 h might be accepted.

[3)]Weight of bare steel

[4)]Batch corresponding to one production unit capacity, weighing not more than 30 tonnes

[5)]Quantity of product generally manufactured in one operation.

A4.6 References of Testing Standards

[S4] - ASTM B117-16 “Standard practice for operating salt spray (fog) apparatus”

[S5]- XP A35-037:2003 “Torons en acier a haute resistance proteges gaines”

[S7]- ISO 15630:2011 “Steel for reinforcement and prestressing of concrete”

[S8]- prEN 10138:2009 “Prestressing Steel”

[S13]- EN 10244:2009 “Steel wire and wire products, non-ferrous metallic coatings on steel wire”

[S14]- ASTM A 882 M-10 “Standard specification for filled Epoxy -coated Seven Wire Prestressing steel strand”

[S16]- ISO 1133:2011 “Plastics-Determination of the melt flow rate (MFR) and the melt volume rate (MVR) of thermoplastics”

[S17]- ISO 1183:2012 “Plastics-Methods for determining the density of non-cellular plastics”

[S18]- ISO 6964:2006 “Polyolefin pipes and fittings-Determination of carbon black content by calcinations and pyrolysis-Test method and base specification”

[S19]- ISO 18553:2002 “Method for the assessment of the degree of pigment or carbon black dispersion in Polyolefin pipes, fittings and compound

[S20]- ISO 527:2012 “Plastics- Determination of tensile properties”

[S22]- ISO 11359-2:1999 “Plastics- Thermo mechanical analysis (TMA)-Part-2: Determination of coefficient pf linear thermal expansion and glass transition temperature”

[S23]- ASTM G14-04 “Standard test method for impact resistance of pipeline coatings (Falling Weight Test)”

[S24]- ISO 2137:2007 “Petroleum products -Lubricating grease and petroleum - Determination of cone penetration”

[S25]- ASTM D217 -16 “Standard test methods for cone penetration of lubricating Grease”

[S26]- ISO 2176:1995 “Petroleum products -Lubricating grease -Determination of dropping point”

[S28]- BS 2000-121:2005 “Method of test for petroleum and its products. Determination of Oil Separation from lubricating grease, Pressure filtration method”

[S29]- DIN 51817:2014 “Determination of Oil Separation from lubricating grease under static conditions”

[S30]- NFT 60 191:2011 “Petroleum products and Lubricating grease- Oil Separation On storage of Lubricating Grease- Method under pressure -Static conditions”

[S31]- ASTM D6184-16 “Standard Test Method for oil separation from Lubricating grease (Conical Sieve Method)”

[S32]- DIN 51808:2015 “Testing of Lubricants; determination of oxidation stability of greases: oxygen method”

[S33]- ASTM D942-15 “Standard Test method for Oxidation Stability of Lubricating Greases by the Oxygen Pressure Vessel Method”

[S34]- ISO 9227:2017 “Corrosion tests in artificial atmospheres-Salt spray tests”

[S35]- ISO 6270-2:2005 “Paints and varnishes-Determination of resistance to humidity - Part-2 Procedure for exposing test specimens in condensation-water atmospheres”

[S36]- ISO 11007:1997 “Petroleum products and Lubricants- Determination of rust prevention characteristics of lubricating greases”

[S37]- NFM 07-023:1969 “Liquid Fuel-Determination of Chlorides in crude petroleum and petroleum products”

[S38]- ISO 2207:1980 “Petroleum waxes-Determination of Congealing point”

[S39]- ISO 2176:1995 “Petroleum products-Lubricant greases-Determination of dropping point”

[S40]- ISO 2592:2000 “Determination of Flash and fire points – Cleveland open cup method”

[S41]- ISO 2137:2007 “Petroleum products and Lubricant - Determination of Cone penetration of lubricating greases and petrolatum”

[S42]- ISO 2160:1998 “Petroleum products- Corrosiveness to copper -Copper strip test”

[S44]- ASTM D1693 “Standard test method for environmental stress cracking of ethylene plastics”

[S45]- ISO 178:2010 “Plastic, Determination of bending modulus”

[S46]- ISO 6295-3:2015 “Thermoplastic pipes-Determination of testing properties-Part 3 - Polyolefin pipes”

[S49]- ASTM G20-10 “Standard Test Method for Chemical resistance of pipeline coatings”

[S50]- ASTM D968-16 “Standard Test Methods for Abrasion Resistance of Organic Coatings”

[S51]- ASTM A370-17 “Standard Test Methods and definitions for mechanical Testing of Steel Products”

[S55]- ISO 22088:2006 “Plastics-Determination of resistance to environmental stress cracking (ECS)”

Annexure A5

(Pertaining to Chapter 13)

Organisation Setup and Training Requirements for Inspection and Maintenance

This Annexure discuss about the minimum organizational setup and training requirements of personnel involved in operation and maintenance. This Annexure has been referred in **Chapter 13**.

A 5.1 Organisation Set up

For the inspection and maintenance of an extradosed bridge project a suitable organization set-up consisting of about 10 to 15 members headed by the Chief Engineer (Maintenance) or of equivalent rank in an O&M Agency outlining the responsibility of each & every staff (see **Table A.5.1**) shall be framed in such a manner that each element of the project shall be checked /inspected properly. The service level of each element of the project shall be assured for safe and undisturbed traffic movement.

Table A 5.1 Organisation Setup for inspection and Maintenance

Chief Engineer (Maintenance)		
Executive Engineer (Maintenance) for Bridges	Executive Engineer (Roads)	Executive Engineer (Maintenance) for Electrical components
Maintenance Engineer (Main EDB/ Approach spans	Maintenance Engineer (Roads)	Maintenance Engineer (Electrical)
Inspector (Bridges	Inspector (Roads)	Inspector (Electrical)

The Chief Engineer (Maintenance), the Executive Engineer (Maintenance), Maintenance Engineer for Bridge and Roads should be qualified civil engineers with at least 20, 15 and 10 years of working experience respectively, major part of which should be in inspection and maintenance of long span bridge structures.

The proposed organization set-up as in **Table A 5.1** may, however, be changed to suit the requirement of maintenance activities at site. The above organization set-up shall be augmented by Supervisors, System-in -Charge, Electrical, Secretary to the Chief Engineer (Maintenance), CAD draftsman, Enumerator, Computer operator and other office assistants as and when required for preparation of drawings, record keeping, estimating and help in other miscellaneous items.

It is essential that the inspection and maintenance team get familiarize themselves thoroughly with design, construction and maintenance of this type of bridges.

Responsibilities

The duties of each inspection and maintenance officers as well assistants are mentioned below.

Chief Engineer (Maintenance) (one no.)

- Organize fortnight meeting to review the maintenance activities.
- Approval of the maintenance activities
- Allotment of fund
- Monitoring the maintenance works.
- Coordination with various agencies like traffic police, local administration, publicity media etc., in case of emergent repairs interruption to traffic by road blockage, etc.
- Monitoring the all-round safety

Duties of Executive Engineer (Maintenance) (one each for bridge, road and Electrical Components if any)

- Conducting special inspection, surveys and site investigations
- Finalization of inspection report
- Checking the cause of defects, structural inadequacy
- Finalization of maintenance scheme & estimates after reporting observations with suggestion for remedial action to Chief Maintenance Manager
- Reporting any damage due to heavy rain, accident, earthquake, cyclone or any type of emergency.
- Supervision of the execution of the maintenance works
- Monitoring & approval of all safety aspects
- Necessary approval of inspection & maintenance works
- Coordination with various agencies like traffic police, local administration specialised execution agencies etc., in case of emergent repairs interruption to traffic by road blockage, etc.
- Finalizing the execution report.
- Preparation of the monitoring report

Duties of Maintenance Engineer (one each for bridge, road, electrical components, ROB)

- Conducting detailed inspection, surveys and site investigations as per approved format
- Prepare inspection report
- Analyse the cause of defects, structural inadequacy
- Prepare maintenance scheme & estimates after Reporting observations with suggestion for remedial action to Executive Maintenance Manager
- Reporting any damage due to heavy rain, accident, earthquake, cyclone or any type of emergency.
- Supervision of the execution of the maintenance works
- Preparation of the execution report.
- Arranging men, materials and machinery in advance as per requirement

Duties of Inspector

- Arranging men, materials and machinery in advance as per requirement
- Conducting detailed inspection, surveys and site investigations as per approved format
- Reporting any damage due to heavy rain, accident, earthquake, cyclone or any type of emergency to the Maintenance Manager.
- Assist the execution of maintenance works

Duties of Supervisor

- Arranging men, materials and machinery in advance as per requirement
- Supervise the execution of maintenance works
- Assist detailed inspection, surveys and site investigations as per approved format
- Reporting to the Maintenance Manager.

Duties of System In-charge

- Routine inspection of the whole system
- Quick action to be taken in case of any type of fault

- Study the condition of whole system
- Supervise the execution of maintenance works
- Reporting to the Maintenance Manager

Duties of Electrician

- Daily inspection of each electrical unit
- Quick action to be taken in case of any type of fault
- Supervise the execution of maintenance works
- Reporting to the Maintenance Manager & System in-charge

A 5.2 Training

Training of all the personnel involved in maintenance is an integral part of the maintenance function. Such training, as may be required by the operating and maintenance personnel to achieve better performance can be classified as below:

- Training of Managers, Inspector and Supervisor.
- Training of Assistant, Electrician and gang men/mates.

(a) Training of Managers, Inspector and Supervisor: These officers are either diploma holders or fresh degree holders. Either way they are expected to perform all the necessary management tasks sometime without any formal management orientation or training. The typical section officer learns about maintenance management while he is on the job. Frequently the new officer handles the job in the same manner as the person before him. Techniques which may or may not have worked twenty years ago are tried again today. Manager and other officers must be provided with the opportunity to attend maintenance management and operations training. The areas to be covered during the training should be

- Identification of defects
- Planning maintenance operations effectively
- Crew scheduling and control to achieve higher productivity at low cost consistent with quality
- Problems of crew supervision—communicating and coordinating
- Preparing road inventory
- Management by objectives
- Repair methods

(b) Training of Assistant, Electrician and gang men/mates: The training of maintenance Assistant, Electrician and gang men/mates is very essential in two areas like (i) Safety aspects and (ii) Repair methods.

The training in safety aspects will include:

- Training of flag-men
- Use and lay out of temporary traffic signs for repair
- Safety oriented handling and parking of machinery and equipment.

The training in repair methods will include:

- Proper patching methods
- Proper grading of shoulders
- Maintenance of drainage structure
- crack sealing
- Painting and maintenance of the traffic signs and warnings
- Equipment maintenance.

Annexure A6

(Pertaining to Chapter 13)

Checklist for Inspection of Stay Cable System (Referred in Chapter 13)

A. Details about the Bridge and Stay Cable System

1) Bridge name and location
2) Year built
3) Main span length (m)
4) Side span length (m)
5) Type of stay cable system used?
6) Type of anchorage used?
7) Do cables go over “saddles” on the tower?
8) Type of cable sheathing used
9) Do the cable systems have viscous dampers installed at deck levels?
10) Do cables have other dampers (other than viscous or neoprene rings) to control vibrations?
11) Do the cables have cross cables installed between them?
12) Whether Coating/treatment has been provided on Cable within free length of cable? No/Yes
13) If Yes, Type of coating/treatment-
14) Are the coatings/treatments on cables discontinued or removed within the anchorage zone? yes, no, not known, not applicable)
15) Whether any of grout has been used? Yes/No
16) If grout has been used, type of grout –
17) Whether any filler material has been used in the anchorage zone- Yes/No
18) If yes, type of filler material used-
19) Have the cables on this bridge (or any of their components) been repaired? If Yes Explain

20) Whether the strands or cables be replaced if needed?

B. Record of distresses noticed during Inspection

21) Have any corrosion noted in Stay cable

22) Have any of the cables subjected to rain-wind-induced oscillation?

23) Has cracking of the cable sheathing or sheathing connections been noted?

a) Any longitudinal cracking of sheathing?

b) Any transverse cracking of sheathing?

c) Any Excessive bulging of sheathing?

d) Any damage of sheathing at the connection of damper or cross-ties?

24) Has cracking or misalignment of the guide pipes been noted?

25) If there is protective tape wrapped around the cable sheathing, whether there is any deterioration of the tape? (tear, crack or delamination)

26) Have any problems associated with neoprene boots been noted?

27) Any cable sag noticed?

28) Identification of damage to connections between anchorage pipes and cable sheathing.

29) Inspection for damage, loosening, lack of water tightness, and deterioration of neoprene boots and band clamps

30) Inspection for damage or dislocation of neoprene rings and keeper rings, if applicable.

31) Identification of gaps between the neoprene rings and the sheathing.

32) Examination of sheathing surface inside the guide pipe through a boroscope or other means, looking for damage or deformation to the sheathing near the anchorage.

33) Review of cracking or damage to guide pipes or evidence of the impact of cable components on guide pipes.

34) Examination of surface conditions on the visible anchorage components including ring nuts, end caps, and bearing plates.

35) Examination of visible parts of saddles for damage, corrosion, and cracking, if applicable.

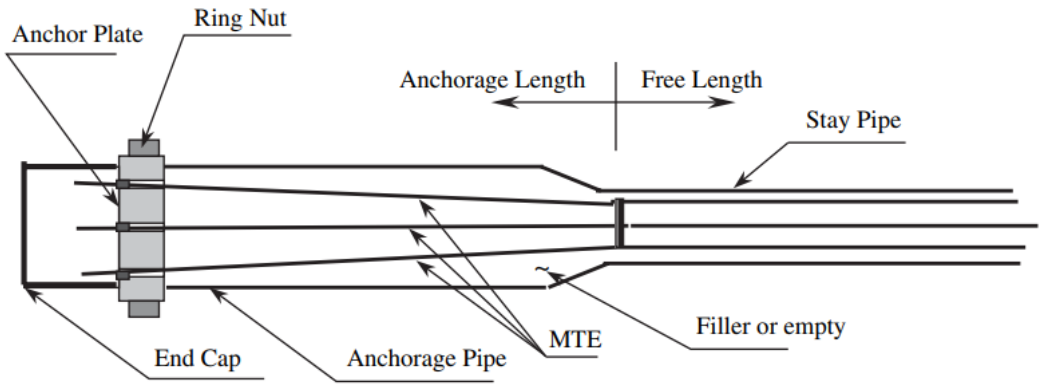
36) Review of evidence of moisture or fillers (such as grease) exiting the anchorage components. If there is an access port at the end cap (ideally at the lowest point), it can be opened and examined for moisture or moisture-contaminated grease

37) After the removal of the end caps on the sockets, visually inspect the anchorage plate and anchorage devices and to see if there is moisture or corrosion inside.

38) Inspection of the cross- tie cables for sagging (losing their force and need to be retensioned).

39) Inspection of damage or cracking on components of cross tie cables. Evidence of fretting and fatigue, especially at connections, are of particular interest.

40) Examination of dampers, if any, as per recommendations of manufacturer



Annexure A7

(Pertaining to Chapter 13)

Design Aspects of Structural Health Monitoring System and Safety Evaluation

This Annexure discusses in detail about the important aspects to be considered while designing a SHM, functional requirement and SHM framework and has been referred in **Chapter 13** of this guideline.

A.7.1 Designing a SHM

The following important aspects shall be considered while designing a SHM

i) When designing a structural health monitoring system, it is fundamentally important to design as an integrated system, which has all data flowing to a single database and presented through a single user interface.

ii) The integration between the different sensing technologies can be simultaneously installed on the structure, for example, electrical strain gauges, fiber optic sensors, vibrating wire sensors, weather stations and corrosion sensors, can be achieved at several levels.

iii) Different sensors can be connected to the same data acquisition system or otherwise, several data-loggers can report to a single data management system, which is generally a personal computer installed either on site or at a remote location.

iv) Even though many of the sensors and systems provide their own software for data management and presentation, one of the goals of the SHM design shall provide a single integrated interface that does not require the end user to learn and interact with different user interfaces.

A.7.2 Functional requirements

There are several functional requirements that must be considered when designing SHM system:

i) The system must implement advanced techniques, good performance, long term stability, and economic value rationality.

ii) The system must have the capability to transmit data, process view, archive document and share long distance information.

iii) The system must be able to collect data synchronously, real-time, long term, and hierarchically.

iv) The system must have capability to assess, control, and calibrates itself.

v) The system must be able to identify damages and evaluate structural health.

vi) The system must be reusable and upgradable

A.7.3 SHM Framework

A Structural Health Monitoring (SHM) System is composed of the following modules:

1) Sensory System
2) Data acquisition and transmission system
3) Data processing and control system
4) Structural health evaluation system
5) Structural health data management system, and
6) Inspection and maintenance system (only if SHM is integrated with inspection and inspection)

Modules 1 and 2 are sensors, data loggers and cabling networks for signal collection, processing and transmission; Modules 3, 4 and 5 are computer systems assigned with different functions of system control and system operation. Module 6 has the purpose of performing the inspection and minor maintenance works for modules 1 and 2.

Sensors provide the basic data for a structural health detection and monitoring system. Their performance directly determines ultimate success of the detection and monitoring methods. Sensors need to record the structural mechanical conditions, and directly transform the measured parameters like strain, displacement, acceleration into signals for output. They must sense the variation of external environmental conditions, which needs sufficient reliability, sensibility and high reaction velocity to reflect the external information promptly and accurately.

Usually, the selection of sensors takes the following factors into consideration: the type, precision, resolution, frequency response and dynamic range; the distribution positions, the influence degree of the surrounding dynamic environment and measurement noise.

For the design of sensor modules and sensing technologies for cable stayed bridges, three aspects should be considered: the parameter to be monitored, the sensor type, and the positioning of the installed sensors.

For the selection of sensor systems, the following aspects should be taken into account:

(i)operational bandwidth; (ii)magnitude and frequency response over that bandwidth; (iii)sensitivity and accuracy; (iv)power supply requirements; (v)physical characteristics (dimensions, weight and material) (vi) environmental operating conditions such as temperature ranges; type of output signal; any signal conditioning requirements; and costs.

The commonly used sensor types and requirements are listed in **Table A7.1**.

Table A7.1 Typical Sensor Types and Requirements

Monitoring Category	Specific Parameter/sensor	Sensor requirement Accuracy, range, resolution, sensitivity
Structural global response monitoring	Acceleration/accelerometer	Large span bridge with low-natural frequency: Frequency response range: 0 ~ 100 Hz Range ≥ ±2 g; transverse sensitivity < 5%
Structural local response monitoring	Strain measurement/ Strain guages	30% full range < static strain measurement < 80% full range Range of dynamic strain gage ≥ 2 × maximum predicted value
	displacement Measurement using displacement sensors	LVDT with measuring ranges between ±1 mm and ±50 mm and a quasi infinite resolution within a temperature range between –20°C and +120°C
	Temperature measurement/ Surface mounted vibrating wire guage	Structural surface temperature: Year extreme minimum temperature −20°C ≤ range ≤ year maximum temperature + 20°C ; Accuracy ≥ ±0.2°C ; resolution ≥ ±0.1°C
Environment monitoring	Wind speed and wind direction monitoring (3 cup Anemometer)	Range ≥ design wind speed of its installation height
	Wind pressure measurement	−1,000 Pa < range < 1,000 Pa Measurement error ± 2Pa
	Atmospheric Temperature monitoring	Atmospheric temperature: Year extreme minimum temperature −20°C ≤ range ≤ year maximum temperature + 20°C; Accuracy ≥ ±0.5°C ; resolution ≥ ±0.1°C
	Humidity measurement	Range: 0–100% RH; accuracy ≥ 3% RH

Data Section

The data section is divided into four parts: collection, transmission, processing, and management. **Table A7.2** deals with the importance aspects to be considered with respect to data related processes.

Table A7.2 Data Requirements

Data	Aspects/parameters	Requirements
Data collection	Selection of equipment	Charge signal: charge amplifier Digital signal: RS485 or compatible with sensor Data collection Analog signal: 4–20 mA and +5 V
	Sampling frequency	Load and environment monitoring: minimum value - Pier acceleration: 50 Hz; wind pressure: 20 Hz; earthquake: 50 Hz; temperature: 1/600 Hz; humidity: 1/600 Hz; rainfall: 1/60 Hz
	Time synchronization Accuracy	Same type of monitoring variables: <0.1 ms Different same type of monitoring variables: <1 ms
Data transmission	Transmission distance	Relatively short and without strong electromagnetic interference: analog signal Far or strong electromagnetic interference: digital signal Up to a few kilometers or even tens of kilometers: optical fiber transmission
Data processing	Pre-processing	Digital filtering, denoising, intercepting
	Post-processing	Specialized design
Data management	Functional requirement	quick display, efficient storage, report generation, and data archiving

A 7.3 Structural Safety Evaluation Requirements

The monitored parameters (load, environment, structural global parameters) can be used for (i) Early Warning and (ii) Safety evaluation of bridge which are discussed below.

Early Warnings

When the structural health monitoring system reports an abnormal situation (as discussed in **Table A.7.3**) to the concerned department, when the global monitoring data exceeds the threshold or limiting value. The threshold value is set according to different levels of danger to the bridge operating environment and structural components

Table A.7.3 SHM data for Early Warning

Safety Warning	Yellow and red warning	(i)Total weight of the vehicle or axle weight > design vehicle load × 1.5 → yellow (>design vehicle load x 2.0 → red) (ii)Maximum average wind speed > design wind speed x 0.8 → yellow (>design wind speed → red) (iii)Maximum/minimum temperature, maximum temperature difference, and maximum temperature gradient > design value → yellow (iv)Displacement > design value x 0.8 → yellow,(>design value or more than 10 yellow warnings appear within a month → red

Safety Evaluations

There are two levels of structural safety evaluation- First level and Second level. Based on the analysis of structural health data collected **Table 11.6** first-level safety evaluation shall be carried out.

In the first level safety evaluation, the eigenvalues of the structure are calculated using the monitoring data, and compared with the predetermined values. If the measured and computed vibration parameters are close, then the bridge is structurally safe. If the measured frequency values are lower than the computed values, then it may indicate a stiffness degradation and thereby indicating the need to perform second-level safety evaluation. Also, if the monitored global response parameters as indicated in **Table A7.4** are abnormal, second-level safety evaluation should be conducted.

Second-Level safety evaluation is a structural safety assessment method in which the corrected structural finite element model, monitored load, and design load are used for structural re-analysis, ultimate load carrying/bearing capacity analysis, and the evaluation of both the bridge structural safety state and the estimated safety reserve.

If the structural local response is found abnormal, special investigations /assessment should be performed. Some of the situations under which special investigations are required are as follows- (i) Measured maximum average wind speed is greater than the design wind speed (ii) barge/ship impact of the bridge (iii) bridge is flooded.

If bridge damage is found during the special assessment, the second-level safety evaluation should be carried out based on the monitored data and special investigations results.

Table A7.4 Global Response Parameters

Structural Health Assessment	Type of Monitoring	Structural performance/ load & environmental Parameters
Data Analysis	Structural global response monitoring	(a) Acceleration: absolute maximum, maximum root mean square value (b) Deformation: mean, absolute maximum (c) Displacement: maximum, cumulative displacement (d) Modal parameters: frequency, mode, and damping ratio
	Structural Local Response Monitoring	Strain: mean, maximum, minimum, stress amplitude maximum; support reaction time course: mean, maximum/minimum, and maximum change
	Load and environment	(a) Vehicle load parameters: bridge traffic, vehicle type, axle weight, gross weight, speed, and overload ratio (b) Wind parameters: speed, direction (c) Earthquake data: acceleration peak, velocity peak, duration, spectrum, and response spectrum (d) Temperature data: maximum/minimum temperature, the maximum temperature gradient of the section (e) Humidity data: maximum internal/external humidity of the component, mean value, and time exceeding the limit (f) Rainfall data: maximum rainfall per hour, cumulative rainfall

CHAPTER 3

References

1. Svensson, Holger; *Cable-Stayed Bridges: 40 Years of Experience Worldwide*, Publs Ernst and Sohn, 2012.
2. *fib* Bulletin 89, *Acceptance of Cable Stayed Systems using Prestressing Steels*, March 2019.
3. PTI DC45.1-18, *Recommendations for Stay Cable Design, Testing and Installation*, Post-Tension Institute, USA, 2018.
4. Setra, *Cable Stays: Recommendation of French Interministerial Commission on Prestressing*, 2002
5. Leonhardt, Fritz: *Bridges Aesthetics and Design.*

CHAPTER 4

References

1. Holger Svensson, *"Cable-Stayed Bridges – 40 years of Experience Worldwide"*
2. Japan Prestressed Concrete Institute "Specification for Design and Construction of Cable-Stayed Bridge and Extradosed Bridge" (2012)

CHAPTER 5

References

1. Svensson, Holger; Cable-Stayed Bridges: 40 Years of Experience Worldwide, Publs Ernst and Sohn, 2012.
2. Gimsing NJ. Cable-supported bridges. Chichester: John Wiley, 1983.
3. R. Walther, B. Houriet, W. Inler and P. Monia, In: (3rd Ed ed.), Cable Stayed Bridges, Telford, London, (1999).
4. Troitsky MS. Cable-stayed bridges. Oxford: BSP Professional Books, 1988.

CHAPTER 6

References

1. Holger Svensenn, Cable stayed bridges, 40 years of experience around the world.
2. Specifications for Design and Construction of Cable-Stayed Bridge and Extradosed Bridge,2012, Japan Prestressed Concrete Institute.
3. Neils J. Gimsing, Christos T.Georgakis, Cable supported bridges, Concept and Design, Third edition, A John Wiley & Sons, Publication.
4. Calvi, G. M., Sullivan, T. J. & Villani, A. (2010). Conceptual Seismic Design of Cable-Stayed Bridges, Journal of Earthquake Engineering, 14:8, 1139-1171, DOI:10.1080/ 13632469.2010.505275
5. El Shenawy EA, Schlaich M. Stay cable forces under dead load for cable-stayed and extradosed bridges," IABSE Symp. Rep., vol. 101, no. 13, p. 1–8, 2013
6. fib Bulletin 89, Acceptance of Cable Stayed Systems using Prestressing Steels, March 2019.
7. Setra, Cable Stays: Recommendation of French Interministerial Commission on Prestressing, 2002
8. Japan Prestressed Concrete Engineering, Japanese Recommendations - Specifications for Design and Construction of Prestressed Concrete Cable-Stayed Bridges and Extradosed Bridges. 2009.
9. Kasuga, Akio, Ikeda S,; "Development of extradosed structures in bridge construction", 25th Conference on Our World In Concrete & Structures: 23 - 24 August 2000.

10. Kasuga, Akio; “Birth, development and future of the extradosed bridge”, Asociación Española de Ingeniería Estructural (ACHE), 2019.
11. “Extradosed Bridges”. IABSE Bulletin 17, 2019.
12. Konstantinos K Mermigas, Behaviour and Design of extradosed bridges, A thesis submitted in conformity with the requirements for the degree of Master of Applied Science Graduate Department of Civil Engineering University of Toronto (2008)
13. Ogawa, A. and Kasuga, A., “Extradosed Bridges in Japan”, FIP Notes, 1998

CHAPTER 7

References

1. Fédération internationale du béton (fib), Bulletin 89, Acceptance of stay cable systems using prestressing steel, 2019.
2. SETRA: CIP Cable stays- Recommendations, 2002.
3. Japan Prestressed Concrete Engineering, Japanese Recommendations - Specifications for Design and Construction of Prestressed Concrete Cable-Stayed Bridges and Extradosed Bridges. 2009.
4. Post-Tensioning Institute (PTI): PTI DC45.1-18. PTI Guide Specifications, Recommendations for Stay Cable Design, Testing and Installation. 2018
5. Akio Kasuga,: Birth, development and future of the extradosed bridge, Hormigon y Acero, 2019.
6. Kasuga, A.: Extradosed bridges in Japan. Structural Concrete, 7(3), 91-103, 2006
7. Svensson, H.: Cable-Stayed Bridges / 40 Years’ Experience Worldwide. Ernst & Sohn, 2012.
8. Gimsing, N.; Georgakis, C.: Cable Supported Bridges, concept and design (Third Edition). John Wiley & Sons, Chichester, UK, 2011.
9. Eurocode 3: Design of steel structures - Part 1-11: Design of structures with tension components.

CHAPTER 8

References

1. Fédération internationale du béton (fib), Bulletin 89, Acceptance of stay cable systems using prestressing steel, 2019.
2. SETRA: CIP Cable stays- Recommendations, 2002.
3. Japan Prestressed Concrete Engineering, Japanese Recommendations - Specifications for Design and Construction of Prestressed Concrete Cable-Stayed Bridges and Extradosed Bridges. 2009.
4. Post-Tensioning Institute (PTI): PTI DC45.1-18. PTI Guide Specifications, Recommendations for Stay Cable Design, Testing and Installation. 2018
5. Svensson, H.: Cable-Stayed Bridges / 40 Years Experience Worldwide. Ernst & Sohn, 2012.
6. Gimsing, N.; Georgakis, C.: Cable Supported Bridges, concept and design (Third Edition). John Wiley & Sons, Chichester, UK, 2011.
7. Eurocode 3: Design of steel structures - Part 1-11: Design of structures with tension components.
8. Tang, M. C., Analysis of cable-stayed bridges, J. Struct. Div. ASCE, Aug., 1971.
9. Tang, M. C., Buckling of cable-stayed girder bridges, J. Struct. Div. ASCE, Sept.,1976.
10. Troitsky, M. S., Cable-Stayed Bridges,Van Nostrand Reinhold, New York, 1988.
11. American Society of Civil Engineers, Guidelines for design of cable-stayed bridges, 1992

12. P. H. Wang, T. C. Tseng and C. G. Yang, Initial Shape of Cable stayed bridges, Computers and structures vol 46, No 6, pp. 1095 – 1106, 1993

CHAPTER 9

References

1. Calvi, G. M., Sullivan, T. J. & Villani, A. (2010). Conceptual Seismic Design of Cable-Stayed Bridges, Journal of Earthquake Engineering, 14:8, 1139-1171, DOI:10.1080/ 13632469.2010.505275
2. Atkins, Jodie C. and Wilson, John C. (2000). Analysis of Damping in Earthquake Response of Cable-stayed Bridges, Proceedings 12WCEE.
3. Kawashima, Kazuhiko and Unjoh Shigeki (1989). Damping Characteristics of Cable-Stayed Bridges Associated with Energy Dissipation at Movable Supports, Structural Eng./Earthquake Eng. Vol. 6. No. 1, 123s-130s. April 1989, Japan Society of Civil Engineers (Proc, of JSCE No. 404/I-11).
4. Wilson, John C. and Atkins, Jodie C. (1999) Damping in Seismic Response of Cable-Stayed Bridges, 8th Canadian Conference on Earthquake Engineering, Vancouver.
5. IRC B3.7 Annexure-F (2022). Draft Guidelines on The Design of Bridge Foundations Subjected to The Effects of Liquefaction of Soil. To be published by IRC Committee B3.

CHAPTER 10

References

1. IRC 6: 2017 “Standard Specification and code of practice for road bridges: Section II-Loads and Load combinations”, Indian Roads Congress
2. IS 875 Part 3: 2015 “Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures - Part 3 : Wind Loads”, Bureau of Indian Standards
3. Holger Svensson, “CABLE-STAYED BRIDGES:40 Years of Experience Worldwide”, Wilhelm, Ernst & Sohn, 2012
4. Lakshmy P et al. “Studies for Aerodynamic stability of Cable Stayed Bridge Decks”, Report of B-25 Research Scheme, Sponsored by MoRTH, CSIR-CRRI, New Delhi, 2007.
5. Leonhardt F and Zellner W (1991) Past, present and future of cable-stayed bridges. In Cable-Stayed Bridges, Recent Developments and their Future – Proceedings of the Seminar, Yokohama, Japan, 10–11 December 1991 (Developments in Civil Engineering) (Ito M, Fujino Y, Miyata T and Narita N (eds)). Elsevier, Amsterdam, the Netherlands.
6. H. Sato, “Wind Resistant Design Manual for Highway Bridges in Japan.”
7. CD 363: Design rules for aerodynamic effects on bridges, Highways, England et. al., 2012
8. Wind-Resistant Design Specifications for Highway Bridges, Professional Recommendatory Standard of Peoples Republic of China , No. JTG/TD 60-01-2004.
9. Abbas, T., Kavrakov, I., and Morgenthal, G.(2017). " Methods for flutter stability analysis of long span bridges: A review ", Proceedings of Institution of Civil Engineers, Bridge Engineering, Vol. 170, Issue BE4, pp. 271-310.
10. FHWA (2012) " FHWA- DESIGN Guidelines for arch and cable-supported Signature Bridges", Report No. FHWA-NHI-11-023.
11. Emil Simiu and DongHun Yeo, “ Modern Structural Design for Wind”, 4th Ed., Wiley-Blackwell, New York, 2019.

12. Yamada, Y. etal (1991). “Earthquake-Resistant and Wind-Resistant Design of Higashi-Kobe Bridge”, Cable Stayed Bridges- Recent Developments and Their Future, pp 397-416.
13. Fujino, Y., Kimura, K., Tanaka, H. (2012). Structural Damping. In: Wind Resistant Design of Bridges in Japan. Springer, Tokyo. https://doi.org/10.1007/978-4-431-54046-5_5
14. Additional reading recommended:
    - SED09: Cable Vibrations in Cable Stayed Bridges, 2007
    - R. Walther, B. Houriet, W. Inler and P. Monia, In: (3rd Ed ed.), *Cable Stayed Bridges*, Telford, London, (1999).
    - N. J. Gimsing, Cable Supported Bridges – *Concept & Design*”, John Wiley & Sons, 2nd Ed. (2012).
    - Krishna, P.,” Aerodynamic Issues in Cable Bridges”, Structural Engineering World Congress, Bangalore,2007.
    - John D. Holmes, “ Wind Loading of Structures”, CRC Press, 3rd Ed.,
15. EC8 (Part-2) (2005). Eurocode 8 - Design of structures for earthquake resistance - Part 2: Bridges. European Committee for Standardization, rue de Stassart, 36 B-1050 Brussels.
16. Der Kiureghian, A. and Neuenhofer, A. (1992). Response spectrum method for multiple-support seismic excitation. Earthquake Engineering and Structural Dynamics 21:7,13-740.

CHAPTER 11

References

1. Stay cables Recommendations of French Inter ministerial Commission on Prestressing (Edition-November 2001 in French, June 2002 in English),SETRA
2. Fib bulletin 89, Recommendation, Acceptance of stay cable systems using prestressing steels (Edition-March, 2019).
3. PTI DC45.1-18 Recommendation for stay cable design, testing and installation (Seventh Edition, November,2018).
4. Paper , Bridge Ice Accretion and De-and Anti-icing System :A Review by Kleissl Kenneth & Georgakis , Christos published in 7th International Cable Supported Bridges Operators Conference-2010
5. IRC:24-Steel Road Bridges (Limit State Method)
6. EN 10337: Zinc and Zinc alloy coated prestressing steel wires and strands
7. ASTM A882/A882M: Standard Specification For Filled Epoxy-coated seven wire prestressing Steel Strand.

CHAPTER 12

References

1. ISIS Canada. (2001). Guidelines for Structural Health Monitoring [Computer File]. Canada: ISIS Canada
2. Mufti, A. A. (2002). Structural health monitoring of innovative Canadian civil engineering structures. Struct. Health Monitor. 1, 89–103. doi:10.1177/147592170200100106
3. Structural Assessment, Monitoring and Control. (2006a). Available at: http://www.samco.org/network/index.htm
4. Yang, Y., Li, Q., and Yan, B. (2017). Specifications and applications of the technical code for monitoring of building and bridge structures in China. Adv. Mech. Eng. 9, 168781401668427. doi:10.1177/168781401668427

5. Brownjohn, J. M. (2007). Structural health monitoring of civil infrastructure. Philos. Trans. R. Soc. London A 365, 589–622. doi:10.1098/rsta.2006.1925
6. Ou, J., and Li, H. (2010). Structural health monitoring in mainland China: review and future trends. *Struct. Health Monitor.* 9, 219–231. doi:10.1177/1475921710365269
7. Moreu, F., Li,X., Li, S., and Zhang, D. (2018) Technical Specifications for Structural Health Monitoring for Highway Bridges: New Chinese Structural Health Monitoring Code, Frontiers in Built Environment, Vol. 10, Article 10, www.frontiers.org.
8. Housner, G.W., Bergman, L.A., Caughey, T.K., Chassiakos, A.G., Claus, R.O., Masri, S.F., Skelton, R.E., Soong, T.T., Spencer, B.F. and Yao, J.T.P. 'Structural Control: Past, Present, and Future,' ASCE Journal of Engineering Mechanics, 123[9], pp. 897–971, 1997.
9. Karbhari, V.M.(2005) 'Health Monitoring, Damage Prognosis and Service-Life Prediction – Issues Related to Implementation,' in Chapter V, Sensing Issues in Civil Structural Health Monitoring, ed. F. Ansari, Springer, pp. 301–310.
10. Karbhari, V.M., and F. Ansari. (2009). "Structural Health Monitoring of Civil Infrastructure Systems." 1st ed. CRC Press
11. Farrar, Charles R., and Keith Worden. (2012). "Structural Health Monitoring: A Machine Leaning Perspective. "http://www.wiley.com/WileyCDA/WileyTitle/productCd1119994330.html.
12. Furtner P., Della CA, and Gosh C., "Structural Health Monitoring of Signature Bridge in Delhi - the Bridge-Structural-Health-Monitoring-System for the Wazirabad Bridge Project"
13. Jang, S., Jo, H., Cho, S., Mechitov, K., Rice, J.A., Sim, S.H., Jung, H.J. Yun, C.B, Spencer, B.F. and Agha, G. (2010) "Structural health monitoring of a cable-stayed bridge using smart sensor technology: deployment and evaluation", Smart Structures and Systems, Vol. 6, No. 5-6, 439-459.
14. Papanikolas, P., Stathopoulos-Vlamis, A. and Panagis, A. (2011) "The structural health monitoring system of Rion Antirion bridge "Charilaos Trikoupis" Proceedings of International Conference on Bridges and Soil-Bridge Interaction, October 13-15, Athens, Greece.
15. Lin H., Xiang Y., Jia Y. (2018) "Study on Health Monitoring System Design of Cable-Stayed Bridge" Facing the Challenges in Structural Engineering. Sustainable Civil Infrastructures. Springer, Cham.