4. DESIGN OF TRAIL BRIDGE

4.1 GENERAL

4.1.1 Loads on Trail Bridge

A cable supported trail bridge is designed with a detailed structural analysis of its major components. The main cable structure is designed for three loading cases, e.g., hoisting, dead load and full load cases. The foundations and the steel parts are designed for the full load case. The wind load is taken into consideration while designing the steel tower and the windguy cable. The loads are assumed static and uniformly distributed along the span.

Live Load

The live load for SSTB and LSTB standards is standardized as such:

For spans up to 50 m, live load, $P = 4.0$ kN/m^2 or 40 kg/m^2
For spans longer than 50 m, $P = 3.0+50/L$ kN/m^2 or $300+5000/L$ kg/m^2

One of the factors governing the cost of a bridge is the live load taken in the bridge design. Proper evaluation of the characteristic live load for a trail bridge is very difficult. The nature of the trail, the traffic volume, the importance of surrounding places, chances of occasional one time big traffic due to festivals, bazaar and many other factors determine the value of the live load. In some cases, the live load, standardized by the above formula, does not conform to reality. But there must be sufficient reason, proven by a statistical analysis, to alter the standard value of the live load in a particular design. Attention should also be given to the resulting light superstructures when applying a light live load. These structures have undesirable side effects like wobbling, swinging, etc.

Dead Load

The dead load includes the weight of all the permanent components of the bridge structure. It is the weight of the cables connected to the walkway structure, the walkway beams, the walkway decks, the hangers and the pretension due to the spanning cables in the N-type. The pretension due to windguy cables is not considered. Care must be taken that the mass (kg, ton) is properly converted into force unit (N, kN) according to the "International System of Units" (1 kg = 9.81N ≈ 10 N or 1 ton = 9.81 kN ≈ 10 kN).

<u>For SSTB Bridge, the dead load excluding handrail and walkway cables is:</u>

Bridge type	Suspended		Suspension
Walkway width	70 cm	106 cm	106 cm
Dead load of walkway excluding walkway and handrail cables	42 kg/m	57 kg/m	79 kg/m
	0.41 kN/m	0.56 kN/m	0.77 kN/m

Hoisting Load

For achieving design cable geometry in dead load, the cable geometry in hoisting case has to be known. Hoisting load means the weight of the cable alone.

Unit weight	Cable diameter in mm				
	13	26	32	36	40
Kg/m	0.61	2.51	3.80	4.81	5.94

Wind Load

The design wind load is assumed to be acting on the bridge walkway only in a horizontal direction to the bridge axis. The other directions are not considered in the design. The dynamic character of the wind load and the vibration of the bridge walkway is a complex calculation and is not applied in the design.

According to the Swiss standard SIA 160, wind pressure is equal to1.3 kN/m^2 for a wind velocity of 39m/s (140 kM/hr). This wind pressure with a coefficient of 1.3 produces a wind load of 0.5 kN per running meter of the LSTB and SSTB standard bridges. It is assumed that the wind exposed area of the standard bridge is 0.3 m^2 per running meter of the bridge. For a bridge site in an exposed area or located high above the water level, it is recommended that a wind load figure of 0.6 kN/m be used in the calculations. See also details in the chapter 4.6.

Snow Load

Snow fall is rare in the mid-hills of Nepal. At high altitudes, the population is small. Hence, there is low probability that the design full load takes place at a time when the bridge is covered by snow. For this reason the snow load is ignored in the bridge design. However, for bridges located at altitudes above 3500 m, investigations on snow loads shall be carried out during the detailed survey.

Temperature Effects

A difference in temperature causes a change in the cable length. A change in the cable length causes a variation in the bridge sag. This variation in sag will be reflected in the cable force. This effect, however, is insignificant and is not considered in the design of the cable. But the changes in the sag needs to be taken into consideration during the hoisting of the cable.

Seismic Load

There is low probability of a full load being on the bridge at the time of an earthquake. Therefore, it is assumed that the seismic load is already covered by the design live load. A separate loading combination with the seismic load is not considered. Nevertheless, it has to be emphasized that the stability of the slopes may be affected by seismic effects, and subsequently can cause damage to the bridge structures.

4.2 WIRE ROPES

The cable (steel wire rope) is the main component of a suspension bridge. The wire ropes used in suspension bridges comply with the following specifications:

IS 1835 – 1977	Steel wire for ropes
IS 6594 – 1977	Technical supply conditions for wire ropes and strands
IS 9282 – 1979	Specifications of wire ropes and strands for suspension bridges
IS 9182/II – 1979	Specifications for lubrication of wire strands and ropes

4.2.1 General Information about Wire Rope

Wire rope is a flexible, multi-wired and stranded precision product. It is composed of wires, strands and a core. In general, any number of multi-wired strands are “laid” or helically arranged around the core.

A predetermined number of wires of proper sizes are helically laid together in a uniform geometrical pattern with a definite pitch or lay. By this process, a strand is formed of the correct diameter. The required number of strands is then closed together around the core with a definite length of lay, forming a wire rope of the required diameter.

The heart of a strand is called the “Centre”.
The heart of the rope is called the “Core”.

The Core may be of three types:-

1. Fibre Core.
2. Steel Strand Core, which may be a wire strand core, abbreviated as WSC (WMC).
3. Independent Wire Rope Core, abbreviated as I.W.R.C. This itself is a small wire rope consisting of 7 strands having 7 wires in each strand.

Fiber cores are not used in suspension bridges. Steel cores are used to obtain:

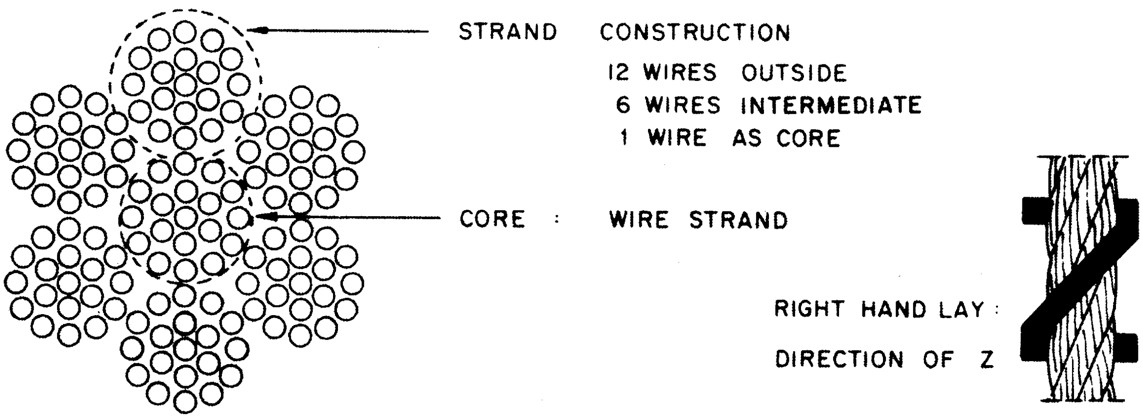
a. Additional strength,

b. Greater resistance to compression or crushing on small drums,

c. Greater ability to withstand shock and impact loads,

d. Less stretch under load.

The illustration given below shows the different components of a wire rope.



WIRE ROPE LAYS

The word "LAY" is used to describe how the wires in the strands and the strands in the rope are laid to form a finished rope. There are two types of lays: 1. ORDINARY LAY, both the wires and the strands are laid, and 2. LANGS LAY.

1. ORDINARY LAY: The wires in the strands and the strands in the rope are laid in opposite directions. If the wires in the strands are laid left hand, and the strands in the rope are laid right hand, it is called RIGHT HAND ORDINARY LAY. In the LEFT HAND ORDINARY LAY, the wires are laid right hand and the strands are laid left hand.

2. LANGS LAY: The wires in the strands and the strands in the rope are laid in the same direction. There are also two types. In the RIGHT HAND LANGS LAY, the wires in the strands and the strands in the rope are laid in the right hand direction. In the LEFT HAND LANGS LAY, the wires in the strands and the strands in the rope are laid in the left hand direction.

The differences in properties are:

The Ordinary Lay	The Langs Lay
Better crushing resistance	More flexible (15 – 20%)
Better resistance to distortion fatigue	More resistant to bending
Less chances of kink formation	Offers more surface area
Less tendency to unwind or rotate under load	More resistant to abrasion
Easier to handle and splice	Will not cause wear on the groove rapidly

CROSS LAID ROPES: Wires of the outer layer cross over the crowns of the underlaying wires which are, therefore, liable to local crushing and cross cutting.

EQUAL LAID ROPES: Wires lie either in a bed formed by the valleys between the wires of an underlayer or alternately along the crown of an underlying wire. These increases wear resistance.

Other things being the same, it is preferable to use ropes in which the wires are of equal lay rather than of cross lay.

PREFORMED WIRE ROPES

In the manufacture of an ordinary wire rope, the first operation is to draw a number of wires through a die giving them a helical or spiral pattern thus forming a strand. A number of strands are then laid together in spiral form to produce a complete rope. In this make-up, there are no compensating factors for the great internal torsion stress built into a Wire Rope. PREFORMING a Wire Rope is the process of reshaping wires and strands into the exact helical positions they will assume in the finished rope, thus relieving the internal stresses in the wire rope.

GALVANISED WIRE ROPES

Galvanizing consists of giving the wires used in the rope a coating of Zinc. The amount of coating depends on the use of the rope. There are two methods of galvanizing: 1. Galvanizing after the finished wire has been drawn, and 2. Galvanizing before the wire is sent for drawing. The specifications for zinc coating are IS: 4826/68 and API-STD.9A. The weights of the coatings are:

Wire ϕmm	**Heavy coatings, gm/m^2 "A-type"**
1.25 – 1.40	180
1.40 – 1.60	190
1.60 – 1.80	200
1.80 – 2.24	210
2.24 – 2.80	230

LUBRICATION

Lubrication prolongs the life of the rope. Suitable lubricants are put on the core, the strands and on the finished rope during manufacture to protect it in transit, during storage and during its working life. It helps to keep the rope free from corrosion. Therefore, the lubricant should contain water repellent and rust preventive additives. In ropes used for Trail Bridges, non-drying and non-bituminous lubricant are used.

KINKS

A kink in a wire rope, no matter how it is made, will damage the strands and wires, thereby greatly reducing the life of the rope. The most common cause of kinks is formation of a loop.

4.2.2 Pre-stretching of Wire Ropes and Elasticity

When a new wire rope is stretched, a certain amount of increase in length takes place. This increment occurs due to its construction and is called construction stretch. This is caused by compression of the core due to the gradual bedding-in of the wires and strands under load. The amount of construction stretch varies and depends upon the size and type of the core, the lengths of both the strands and the rope lays, the construction of both the strands and the rope, the amount and type of load on the rope and the amount of bending to which the rope is subjected. For ropes with steel cores, the stretch is 0.5 to 1.0%.

Pre-stretching is a process of cyclic loading of the rope between 10% and 50% of the minimum breaking load until the virtual elimination of "Initial or Construction stretch" is achieved. A pre-stretched wire rope has a definite and known Modulus of Elasticity.

LSTB AND SSTB WIRE ROPES

The wire ropes used in the LSTB and SSTB trail bridges are such:

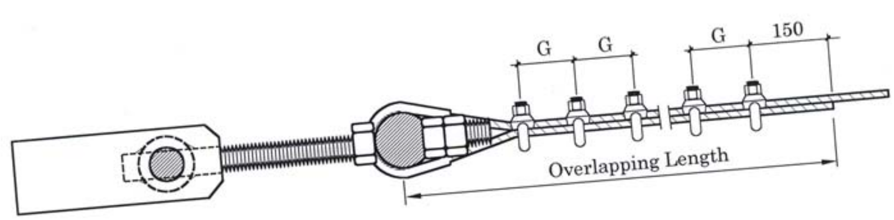
<table>
<tr><th rowspan="2">Cable size, ϕ mm</th><th rowspan="2">Pre-stretched</th><th rowspan="2" colspan="2">Cable size used in</th><th rowspan="2">Number of wires</th><th>Metallic area</th><th>Unit mass</th><th>Min .Breaking Load</th><th>Perm. Load</th><th>Remarks</th></tr>
<tr><th>mm²</th><th>Kg/m</th><th>kN</th><th>kN</th><td rowspan="6">Tensile strength of wire = 1.57 (kN/mm²)

Modulus of Elasticity, E = 110 (kN/mm²)</td></tr>
<tr><td>13</td><td>Non</td><td rowspan="3">SSTB</td><td rowspan="3">LSTB</td><td>7 x 7 (6 x 1)</td><td>73</td><td>0.64</td><td>103</td><td>34</td></tr>
<tr><td>26</td><td rowspan="4">Yes</td><td rowspan="4">7 x 19 (12 x 6 x 1)</td><td>292</td><td>2.51</td><td>386</td><td>129</td></tr>
<tr><td>32</td><td>442</td><td>3.80</td><td>585</td><td>195</td></tr>
<tr><td>36</td><td colspan="2" rowspan="2">LSTB</td><td>560</td><td>4.81</td><td>740</td><td>247</td></tr>
<tr><td>40</td><td>691</td><td>5.94</td><td>914</td><td>305</td></tr>
</table>

All cables are: Right Ordinary Lay (RHO), Wire Strand Core, construction same as other strand (WRC), Heavy "A" Galvanizing, Preformed and with non-drying and non-bituminous lubricant.

ROPE FITTING

In LSTB and SSTB practices, the rope ends are secured by bending the tail of the rope a round a terminal or a beam by means of Bulldog grips. If correctly used, the fitting is approximately 90% as rope. The "D" of the bolt must be applied on the dead end of the rope. The base or the casting of the grip should be on the main part of the rope which takes the load, and the "U" on the short tail end. The reason being that the "U" will always distort the rope underneath due to its round shape, and if applied on the main rope, its strength is reduced due to its crushing effect. The methods are shown below:



4.3 CONSTRUCTION MATERIALS

The construction materials used in the standard trail bridges, besides the above mentioned wire ropes (cables), are steel parts, steel fixtures and fasteners (thimbles, bulldog grips, nuts and bolts), concrete and stone masonry. The specifications of these materials are based on the Indian Standard (IS). Refer to Volume A of the LSTB Manual for detailed material specifications. However, for frequently mentioned material, the specifications are given below for quick reference.

4.3.1 Structural Steel

Steel grade should be of standard quality **Fe 410** and structural steel should comply with the requirements of:

IS 226 – 1975	Structural Steel
IS 800 – 1984	General Construction Steel

The steel should have the following mechanical properties:

Tensile Strength	≥	410 N/mm^2
Yield Stress	≥	250 N/mm^2
Modulus of Elasticity	=	200 kN/mm^2
Elongation	≥	23%

4.3.2 Fasteners

Bolts, nuts and washers should be of grade C, property class 4.6 and should comply with the requirement of:

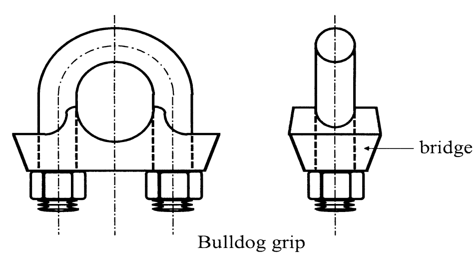
IS 1363 - 1984 (Part 1)	Hexagonal Head Bolts and Nuts
IS 1367 – 1983	Threaded Fasteners

All the fasteners should be hot dip galvanized with a minimum zinc coating of **40μm**.

4.3.3 Bulldog Grips

Bulldog Grips are used at the cable terminals to secure the cable ends.

Bulldog Grips should conform to IS 2361-1970 specifications for Bulldog Grips. The bridges must be drop-forged and suitably scored to grip a round strand rope of right hand lay having six strands. Bridges, U-bolts and nuts should be hot dip galvanized with a minimum zinc coating of 40μm. The size of the bulldog grip is equal to the size of the cable to be anchored or connected.



4.3.4 Reinforcement Steel

Reinforcement Steel should be of steel grade **Fe 415**, high yield deformed bars and should comply with the requirements of:

IS 1786 – 1985	High Strength Deformed Steel Bars for Concrete Reinforcement
IS 456 – 1978	Plain and Reinforced Concrete

The Reinforcement Steel should have the following mechanical properties:

Yield Stress	=	415 N/mm^2
Modulus of Elasticity	=	210 kN/mm^2

4.3.5 Rust Prevention

Rust prevention of steel parts should be done by **Hot Dip Galvanization** according to:

IS 2629 – 1966 Recommended Practice for Hot Dip Galvanization of Iron and Steel
IS 4759 – 1984 Specification for Hot Dip Zinc Coating on Structural Steel

Products	Minimum Mass of Coating		Minimum Thickness of Coating
	g/m²	N/m²	µm
Structural Steel	610	6.0	80
Threaded work, Nuts and Bolts	300	3.0	40

4.3.6 Wire Mesh Netting

Wire for wire mesh netting should comply with the requirements of:

IS 280 - 1978 Mild Steel Wire for General Engineering Purposes
IS 4826 - 1979 Hot Dip Galvanizing Coatings on Round Steel Wire

The diameter of the wire should be 12 SWG (2.64 mm) and the zinc coating should not be less than 270 g/m^2. The average tensile strength of the wire should not be less than 380 N/mm^2.

4.3.7 Concrete

Concrete should comply with all the requirements of:

IS 456 – 1978 Plain and Reinforced Concrete
IS 269 – 1989 Ordinary Portland Cement
IS 383 - 1970 Coarse and Fine Aggregate

Concrete Grades used in the standard design are:

Concrete 1:3:6 (M10) for miscellaneous use
Concrete 1:2:4 (M15) for structure

4.3.8 Masonry

Masonry should comply with all the requirements of:

IS 1597 – 1967 Code of Practice for Construction of Stone Masonry
IS 2250 – 1981 Preparation and Use of Masonry Mortars

Stone masonry used in the standard design are:

Chisel Dressed Stone Masonry in 1:4 cement : sand mortar
Hammer Dressed Stone Masonry in 1:6 cement : sand mortar
Dry Stone Masonry

4.3.9 Unit Weight of Construction Materials

The unit weights of the construction material used in the standard design is given in the following table.

Materials	Unit Weight	
	kg/m³	kN/m³
Concrete	2200	22.0
Stone Masonry	2100	21.0
Steel	7850	78.5
Soil	1800	18.0
Water	1000	10.0

4.4 CABLE DESIGN

Designing of the cable in a suspension/suspended trail bridge is done to determine the safe cable size and number for a full load condition based on the pre-defined cable geometry for the dead load condition. Also, the design calculation predicts the cable geometry for the hoisting case to achieve the defined cable geometry in the dead load.

There are three different methods of cable analysis:

1. Analytical analysis using the deflection method
2. Finite deflection analysis
3. Finite element analysis

4.4.1 Cable Geometry and Methods of Cable Analysis

CATENARY CURVE OF CABLE

When a perfectly flexile uniform cable hangs freely between two fixed points not in the same vertical line, the curve, in which it hangs under the action of gravity is called the ***catenary***. If the weight per unit length of the cable is constant, the catenary is called uniform or common catenary. Perfectly flexible, means that the cable offers no resistance to bending at any point. In such cases, the resultant action across any section of the cable consists of a single force whose line of action is along the tangent to the curve formed by the cable.

Let a cable suspended between points **A** and **B** have the lowest point at **O**. Let the length of the cable along the curve measured from point **O** to point **B** be ***S***. If the weight of the cable per unit length is g_h, the weight of the section **OB** will be g_h **S**.

Consider that the section **OB** is in equilibrium. This section is subject to horizontal tension **H** at **O**, tension **T** at **B** and its weight g_h **S**. The three forces are lying in one plane. The line of action of the weight must pass through the point where the lines of action of **H** and **T** meet. Resolving the forces horizontally and vertically, we have:

$$T \cdot Cos\theta = H$$
$$T \cdot Sin\theta = g_h s$$

and $Tan\theta = \dfrac{g_h}{H} \cdot S$

It is convenient to write $H = g_h C$,
Then, $S = C \ Tan\ \theta$. This is called the basic equation of the catenary. And the constant C is known as the parameter of the catenary.

It can be written:

$$\frac{dy}{dx} = Tan\theta = \frac{S}{C} \Rightarrow S = c\frac{dy}{dx}$$

Differentiating

$$\frac{ds}{dy} = C\frac{d^2y}{dx} \Rightarrow \sqrt{\left[1+\left(\frac{dy}{dx}\right)^2\right]} = C\frac{d^2y}{dx}$$

Let $p = \dfrac{dy}{dx}$

We get $\sqrt{\left(1+p^2\right)} = C\dfrac{dp}{dx}$

On integration, we get: $x + A = C \sinh^{-1}(p) = C \sinh^{-1}\left(\frac{dy}{dx}\right)$

Here, A is a constant of integration.
We can choose conveniently the coordinate axis. If we take the vertical through the lowest point of the catenary as the axis of y, we have;

$\frac{dy}{dx} = 0$, when x =0. This makes A =0. Hence $\frac{dy}{dx} = \sinh\frac{x}{C}$

Integrating again, we have $y = C\cosh\frac{x}{C} + B$ Here, **B** is a constant of integration.

If we take the origin at a depth ***C*** below the lowest point of the catenary, we have:
y = C, when x = 0, giving B = 0.

Hence $y = C\cosh\frac{x}{C}$

This gives the shape of the curve adopted by the cable. When required, the length of any arc of the cable is:

$$S = C\frac{dy}{dx} = C\sinh\frac{x}{C}.$$

The equation, H = g_h ***C***, shows that the tension at the lowest point is equal to the weight of the cable whose length is the same as the distance between the origin and the vertex.

As for the tension in the cable at **B**, we have:

$$T^2 = g_h^2\left(s^2 + C^2\right),$$

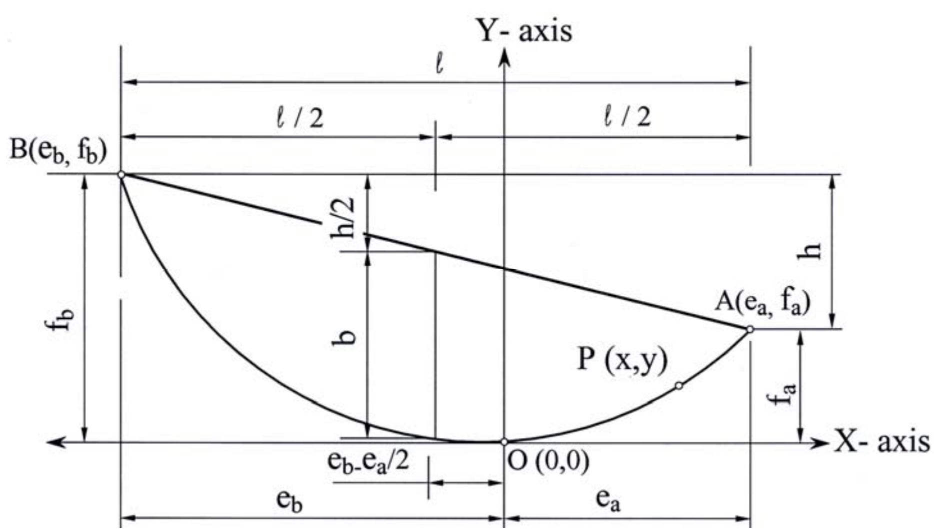
$$T^2 = g_h^2 y^2$$

$$T = g_h y$$

Hence, the tension varies as the height above the directrix.

We may, therefore, note that

a) The horizontal component **H** is constant and equal to $\mathbf{g_h.C}$
b) The vertical component of **T** at point **B** is equal to $\mathbf{g_h.S}$
c) The resultant tension **T** at point **B** is equal to g_h .y, where y is the height above the directrix



PARABOLIC CURVE OF CABLE

In many practical cases, the self-weight of the cable forms only a small fraction of the total imposed loads. Hence, for analysis, it may be approximated to a parabola if subjected to uniformly distributed loads on plan. The analysis of cable geometry in parabolic shape is simple and in actual practice more realistic because of:

1) The cable, particularly in a suspension bridge, has the loads, instead of being distributed as though uniform along the cables, more nearly distributed uniformly along the span length.

2) The difference in shape between a parabola and a catenary with the same span and same central sag is only about 0.04% and 0.10% for the sag/span ratio of 0.10 and 0.20. Hence the difference in the cable geometry is almost negligible for the sag/span ratio of practical range. The assumed shape of a parabola influences very little the calculation of the suspenders and the tensions in the cable.

We will consider a cable, as before, to be perfectly flexible and inextensible. The cable is hung between two fixed points **A** and **B**. Let **O** be lowest point of the cable and be taken as the origin of the coordinates, **X** axis as abscissa and **Y** axis as ordinate. The load is constant and is uniformly distributed per unit of span length. Let us take a part of the cable **OPB**. The part is in equilibrium. There are two axial forces acting on their ends. Their directions of action are along the tangent to the curve formed by the cable. We will take **H** as the tension in the cable at **O** and **T** as that at the point **B**. By the theory of static, for a system in equilibrium,

a) the sum of the forces projected along the axis should be zero

$$\sum X = 0$$

$$\sum Y = 0$$

b) the sum of the moments of all forces at any point should be zero

$$\sum M = 0$$

Let the coordinate of the point **P** be (**x , y**). Separate a part OP of the cable. There are two axial forces **H** and **T$_x$** acting on their ends. If the load on the cable is taken as **g** per unit span length, the part **OP** will have a vertical load of, $\boldsymbol{g \cdot x}$ acting downward at the distance $\frac{x}{2}$ from the origin.

The sum of the moment at **P**:

$$\sum M_P = H \cdot y - \frac{gx^2}{2} = 0$$

$$y = \frac{gx^2}{2H}$$

$$\sum X = H_x = T_x \cos \beta_x - H = 0 \Rightarrow T_z \cos \beta_x = H \Rightarrow\Rightarrow T_x = \frac{H}{\cos \beta_x}$$

$$\sum Y = V_x = T_x \sin \beta_x - gx = 0 \Rightarrow\Rightarrow T_x \sin \beta_x = gx \Rightarrow\Rightarrow V_x = gx$$

$$T_x^2 \left(\sin^2 \beta_x + \cos^2 \beta_x\right) = H^2 + (gx)^2 \Rightarrow\Rightarrow T_x = \sqrt{H^2 + (gx)^2}$$

$$T_x = H\sqrt{1+\left(\frac{gx}{H}\right)^2} = H\left(1+\frac{1}{2}\left(\frac{gx}{H}\right)^2\right) = H+\frac{1}{2}\frac{g^2x^2}{H} = H+\frac{1}{2}\frac{g^2x^2}{\frac{gx^2}{2y}} = H+gy$$

$$T_x = H + g.y$$

The span between **A** and **B** is ***l*** and the level difference is **h**
Let the coordinates of the points A and B be (e_a, f_a) and (e_b, f_b), then we have:

$$e_a + e_b = l \text{ and } f_b - f_a = h$$

$$f_b = \frac{ge_b^2}{2H}$$

$$f_a = \frac{ge_a^2}{2H} \qquad \text{and} \quad h = \frac{g}{2H}\left(e_b^2 - e_a^2\right) = \frac{g}{2H}\ell\left(2e_b - \ell\right) \Rightarrow e_b = \frac{\ell}{2}+\frac{Hh}{g\ell}$$

$$\frac{f_b}{f_a} = \frac{e_b^2}{e_a^2} \Rightarrow \frac{e_b}{e_a} = \frac{\sqrt{f_b}}{\sqrt{f_a}}$$

$$\frac{e_b}{e_a+e_b} = \frac{\sqrt{f_b}}{\sqrt{f_a+\sqrt{f_b}}}$$

$$e_b = \ell\frac{\sqrt{f_b}}{\sqrt{f_a+\sqrt{f_a}}}$$

$$e_a = \ell\frac{\sqrt{f_a}}{\sqrt{f_a+\sqrt{f_b}}}$$

$$H = \frac{ge_b^2}{2f_b} = \frac{g\ell^2 f_b}{2f_b\left(\sqrt{f_a}+\sqrt{f_b}\right)^2} = \frac{g\ell^2}{8\frac{\left(\sqrt{f_a}+\sqrt{f_b}\right)^2}{4}} = \frac{g\ell^2}{8b}$$

Here, $\quad b = \frac{\left(\sqrt{f_a}+\sqrt{f_b}\right)^2}{4}$

Check if the sag of the cable at the middle of the span is equal to **b**

$$b = f_b - \frac{h}{2} - \frac{f_b}{e_b^2}\left(e_b - \frac{\ell}{2}\right)^2 = f_b - \frac{f_b - f_a}{2} - \frac{f_b}{e_b^2}\left(\frac{2e_b - e_b - e_a}{2}\right)^2 = \frac{f_b + f_a}{2} - \frac{f_b}{4}\left(1-\frac{e_a}{e_b}\right)^2$$

$$b = \frac{f_a + f_b}{2} - \frac{f_b}{4}\left(1-\frac{\sqrt{f_a}}{\sqrt{f_b}}\right)^2 = \frac{f_a+f_b}{2} - \frac{f_b + f_a - 2\sqrt{f_a}\sqrt{f_b}}{4} = \frac{\left(\sqrt{f_a}+\sqrt{f_b}\right)^2}{4}$$

Hence, it is proved that b is the sag at the middle of the span and $H = \frac{g\ell^2}{8b}$

Now, in terms of mid sag ***b***, the relations are:

$$e_b = \frac{\ell}{2} + \frac{hH}{g\ell} = \frac{l}{2} + \frac{h\ell}{8b} = \ell\left(\frac{4b+h}{8}\right)$$

$$e_a = \ell\left(\frac{4b-h}{8}\right)$$

$$f_b = \frac{ge_b^2}{2H} = \frac{4b}{\ell^2}e_b^2 = \frac{(4b+h)^2}{16b}$$

$$f_a = \frac{(4b-h)^2}{16b}$$

The angles are:

At any point, say at **P** (x, y) is

$$Tan\beta_x = \frac{dy}{dx} = \frac{d\left(\frac{g}{2H}x^2\right)}{dx} = \frac{2y}{x}$$

The angles at points **A** and **B** are:

$$Tan\beta_a = \frac{2f_a}{e_a} = 2\frac{(4b-h)^2}{16b}\frac{8}{(4b-h)l} = \frac{4b-h}{\ell}$$

$$Tan\beta_b = \frac{2f_b}{e_b} = \frac{4b+h}{\ell}$$

The forces at A and B are:

$$V_a = ge_a$$
$$V_b = ge_b$$

$$T_a = \frac{H}{\cos\beta_a} = \sqrt{H^2 + g^2e_a^2} = H\sqrt{1+64\left(\frac{be_a}{\ell^2}\right)^2} \approx H + g.f_a$$

$$T_b = \frac{H}{\cos\beta_b} = \sqrt{H^2 + g^2e_b^2} = H\sqrt{1+64\left(\frac{be_b}{\ell^2}\right)^2} \approx H + g.f_b$$

If we shift the origin of the parabola to point **B** with the directions of axis **X** and axis **Y** as shown in the figure, the equation of the parabola is:

$$f_b - y = \frac{g}{2H}(e_b - x)^2 \Rightarrow y = \frac{g}{2H}e_b^2 - \frac{g}{2H}(e_b - x)^2 = \frac{g}{2H}\ell(2e_b - x) = \frac{g}{2H}\ell\left(\frac{2Hh}{g\ell} + \ell - x\right)$$

$$y = \frac{g}{2H}x(\ell - x) + x\frac{h}{\ell} = \frac{g}{2\frac{g\ell^2}{8b}}x(l - x) + \frac{h}{\ell}x$$

$$y = \frac{4b}{\ell^2}(l - x)x + \frac{h}{l}x$$

$$f_x = \frac{4b}{\ell^2}(\ell - x)x$$

The total length is:

$$L = \int_0^\ell ds = \int_0^\ell \left[1 + \left(\frac{dy}{dx}\right)^2\right]^{\frac{1}{2}} dx$$

$$L = \int_0^\ell \left[1 + \frac{1}{2}.1^{\frac{1}{2}-1}.\left(\frac{dy}{dx}\right)^2 + \frac{\frac{1}{2}\left(\frac{1}{2}-1\right)}{2!}.1^{\frac{1}{2}-2}.\left(\frac{dy}{dx}\right)^4 - \frac{\frac{1}{2}\left(\frac{1}{2}-1\right)\left(\frac{1}{2}-2\right)}{3!}.1^{\frac{1}{2}-3}.\left(\frac{dy}{dx}\right)^6 ......\right]$$

Neglecting the minor part, we get

$$L = \int_0^\ell \left[1 + \frac{1}{2}\left(\frac{dy}{dx}\right)^2\right]dx = \int_0^l \left[1 + \frac{1}{2}\left(\frac{4b(\ell - 2x) + h\ell}{\ell^2}\right)^2\right]dx = \ell\left[1 + \frac{8}{3}\left(\frac{b}{\ell}\right)^2 + \left(\frac{h}{\ell}\right)^2\right]$$

Here, $\dfrac{dy}{dx} = \dfrac{4b(\ell - 2x)}{\ell^2} + \dfrac{h}{\ell}$

For plotting the parabola in a graphic method, exercise in following steps:

1. Fix the origin of the coordinate at the lowest point "O" of the parabola.
2. On the X axis, mark any number of intervals, say "n" each interval equal to $\dfrac{e}{n}$.
3. Draw a line EB parallel to the Y axis at the distance "***e***" from the origin.
4. Mark "n" intervals on the line EB, each interval equal to $\dfrac{f}{n}$.
5. Draw lines from the origin to the points marked on the line EB.
6. Mark the points intersected by the vertical and inclined lines as shown in the figure.
7. Joint these points and get the required parabola.

The tension at any point of the parabola is directed along the tangent.

4.4.2 Sag Calculation

The sag calculation of a cable structure is important for determining:

1. The hoisting sag of the cable for achieving the pre-defined dead load sag, corresponding to the whole dead load in the cable structure (all loads connected to the main cable under study),
2. The full load sag for calculating the tensions at both cable ends due to the increment of the dead load to full load in the cable structure (dead load + live load).

4.4.2.1 Basic Principles

It is necessary to understand the following basics:

Non-linear behavior of cable

A system in equilibrium at a given cable geometry and load combination yields certain stress (tension) in the cable. Its axial value at any point along the cable is defined. When there is a definite change in the load combination, the given cable geometry will also change. But the change in the geometry cannot be known by the linear equation of elasticity. The cause is that once the geometry is changed, the stress in the cable will consequently change. The change in the cable length due to the changed cable geometry on one side, and the change in the cable length due to the elastic effect on it by the changed stresses on the other side, must match. This procedure is done in an iterative way.

Change in cable length, fixed to rigid supports due to change in sag

Assume a cable is extended between two fixed rigid supports with known cable geometry as in a suspended bridge. The relation between the changes in the total length, **ΔL** in respect to the difference in sag, **Δb** will be as such:

For a parabolic geometric of the cable, the total length in respect to the sag and the span is calculated as such:

$$L = \int_0^{\ell} ds = \int_0^{\ell} \left[1 + \left(\frac{dy}{dx} \right)^2 \right]^{\frac{1}{2}} = \ell . \left[1 + \frac{8}{3} \left(\frac{b}{\ell} \right)^2 - \frac{32}{5} \left(\frac{b}{\ell} \right)^4 + ... \right]$$

Partial differentiation of this equation with respect to the sag, **b** gives:

$$\frac{\Delta L}{\Delta b} = \frac{16}{15} \left(\frac{b}{\ell} \right) \left[5 - 24 \left(\frac{b}{\ell} \right)^2 \right]$$

Let, $aa = 16 \, \frac{b}{\ell} \left[5 - 24 \left(\frac{b}{\ell} \right)^2 \right]$ Then, $\Delta b = \frac{15 \Delta L}{aa}$

Change in cable length, fixed to flexible supports due to change in span

Assume a cable is extended between two flexible supports with known cable geometry as in a suspension bridge. By flexible supports, it is understood that the saddles may move and the span can alter from its initial value. The relation between the changes in total length, **ΔL** in respect to the difference in sag, **Δb** and in respect to the difference in span, **Δl** will be as such:

For a parabolic geometric of the cable, the total length in respect to the sag and the span is calculated as such:

$$L = \ell.\left[1+\frac{8}{3}\left(\frac{b}{\ell}\right)^2 - \frac{32}{5}\left(\frac{b}{\ell}\right)^4 + ...\right]$$

Partial differentiation of this equation with respect to the span, ***l*** gives:

$$\frac{\Delta L}{\Delta \ell} = 1-\frac{8}{3}\left(\frac{b}{\ell}\right)^2 + \frac{96}{5}\left(\frac{b}{\ell}\right)^4 = \frac{1}{15}\left[15+40\left(\frac{b}{\ell}\right)^2 + 288\left(\frac{b}{\ell}\right)^4\right]$$

Let, $bb = 15+40\left(\frac{b}{\ell}\right)^2 + 288\left(\frac{b}{\ell}\right)^4$

Then, $\Delta L = \frac{bb}{15}\Delta\ell$

The relation between the span and the sag:

$$\Delta b = \frac{15\Delta L}{aa} = \frac{bb\ \Delta\ \ell}{aa}$$

The stress along the cable is variable.

The tension at any point with coordinate (x, y) of the cable parabola is:

$$T_x = \frac{H}{\cos\beta_x} \approx H + gy$$

The average tension (average stress) in the cable:

$$\overline{T} = \frac{\int_{-\frac{\ell}{2}}^{+\frac{\ell}{2}}\left(H+g\frac{g}{2H}x^2\right)dx}{\int_{-\frac{\ell}{2}}^{+\frac{\ell}{2}}dx} = \frac{\int_{-\frac{\ell}{2}}^{+\frac{\ell}{2}}\left(H+g\frac{4b}{\ell^2}x^2\right)dx}{\ell} = \frac{H\ell + g\frac{4b}{\ell^2}\frac{\ell^3}{3}}{l} = H + \frac{4}{3}gb$$

The elastic elongation of the cable due to change in loadings

Hook's Law states:
the ratio of $\frac{Stress}{Strain}$ is defined by the Modulus of Elasticity, E.

Suppose a cable in a dead load parabola has a length equal to L_d. Now the unit load is changed from its initial value $\boldsymbol{g_d}$ to $\boldsymbol{g^*}$. The parabola has taken a new shape due to the new load. The sag is changed. The total cable length is changed.

The ratio of change in the total length to initial total length, i.e., strain is defined by the difference in initial stress. Mathematically, it is as such:

$$\frac{\overline{T_*} - \overline{T_d}}{A} = \frac{L_* - L_d}{L_d} E \Rightarrow\Rightarrow \frac{\Delta \overline{T}}{A} = \frac{\Delta L}{L_d} E$$

$$\overline{T_*} = H_* + \frac{4}{3} g_* b_* \qquad \& \qquad \overline{T_d} = H_d + \frac{4}{3} g_d b_d$$

$$\Delta \overline{T} = H_* - H_d + \frac{4}{3}\left(g_* b_* - g_d b_d\right) = \frac{g_* \ell^2}{8b_*} - \frac{g_d \ell^2}{8b_d} + \frac{4}{3}\left(g_* b_* - g_d b_d\right) = g_*\left(\frac{\ell^2}{8b_*} + \frac{4b_*}{3}\right) - g_d\left(\frac{\ell^2}{8b_d} + \frac{4b_d}{3}\right)$$

The change in the cable length can also be shown by geometric relations:

$$\Delta L = L_* - L_d = \ell\left[1 + \frac{8}{3}\left(\frac{b_*}{\ell}\right)^2\right] - \ell\left[1 + \frac{8}{3}\left(\frac{b_d}{\ell}\right)^2\right] = \frac{8}{3\ell}\left(b_*^2 - b_d^2\right)$$

Equating the change in the cable length calculated by geometric relations and Hook's formula, we get:

$$g_*\left(\frac{\ell^2}{8b_*} + \frac{4b_*}{3}\right) - g_d\left(\frac{\ell^2}{8b_d} + \frac{4b_d}{3}\right) = \frac{8\left(b_*^2 - b_d^2\right)}{3\ell L_d} AE$$

$$g_*\left(\frac{\ell^2}{8b_*} + \frac{4b_*}{3}\right) = g_d\left(\frac{\ell^2}{8b_d} + \frac{4b_d}{3}\right) + \frac{8\left(b_*^2 - b_d^2\right)}{3\ell L_d} AE$$

$$g_* = g_d \frac{b_*}{b_d}\left(\frac{3\ell^2 + 32b_d^2}{3\ell^2 + 32b_*^2}\right) + \frac{64AE}{\ell L_d} b_*\left(\frac{b_*^2 - b_d^2}{3\ell^2 + 32b_*^2}\right)$$

In up-to-date practices, the stress in the cable is determined by the horizontal stress, $\frac{\Delta H}{A}$ at the lowest point. Then the above formula is simplified as:

$$\Delta H = \frac{\Delta L}{L_d} EA$$

$$g_* = \frac{64EA}{3\ell^3 L_d} b_*\left(b_*^2 - b_d^2\right) + \frac{b_*}{b_d} g_d$$

The difference in the calculation in finding the hoisting or full load sags from these is negligible.

4.4.2.2 Sag Calculation of a Suspended Bridge

In a suspended bridge, all the load bearing cables are suspended with the same sag (dip). The sag ranges from $\frac{\ell}{23}$ to $\frac{\ell}{19}$ in the LSTB design standard. The level difference, h between the two saddles is permitted below the value, $\frac{\ell}{14}$ i.e. $h \le \frac{\ell}{14}$. The sag in the dead load condition is defined in consideration of the entrance angle at the higher saddle. The entrance angle at the higher saddle is not permitted to be above 12°. The concept is that movement at the entrance part will be difficult if the entrance angle is high. The sag in dead load case in the LSTB standard is variable and lies as such:

$$b_d^{\min} = \frac{l}{23} - \frac{h}{4} \Rightarrow\Rightarrow \beta_d^{\min} \le 10^o \text{ and } b_d^{\max} = \frac{\ell}{19} - \frac{h}{4} \Rightarrow\Rightarrow \beta_d^{\max} \le 12^o$$

If the sag increases the maximum tension in the cable decreases making the whole bridge design more economical. However, maximum sag should be within the limit of β_{max}.

In the SSTB standard, the dead load sag is fixed and taken as such:

For span, $\ell \le 80.0m, \Rightarrow\Rightarrow b_d = \frac{\ell}{20} \quad and \quad h = \frac{\ell}{25}$

For span, $\ell > 80.0m, \Rightarrow\Rightarrow b_d = \frac{\ell}{22} \quad and \quad h = \frac{\ell}{25}$

In order to obtain the hoisting and full load sags, $\boldsymbol{b_h}$ and $\boldsymbol{b_f}$, corresponding to the hoisting and full unit loads, $\boldsymbol{g_h}$ and $\boldsymbol{g_f}$, respectively, the following procedure of iteration is carried out:

0. Assume: $b_1 = 1.22.b_d$ for full load sag ;
   $b_1 = 0.93.b_d$ for hoisting load sag
1. Start with *i =1, 2, 3…..*
2. Calculate: $L_d = \ell.\left[1 + \frac{8}{3}\left(\frac{b}{\ell}\right)^2 + \frac{1}{2}\left(\frac{h}{\ell}\right)^2\right]$
3. Calculate: $g_i = \frac{64EA}{3\ell^3 L_d} b_i\left(b_i^2 - b_d^2\right) + \frac{bi}{b_d} g_d$
4. Calculate: new $b_{i+1} = b_d + \left(b_i - b_d\right). \frac{g_h\left(g_f\right) - g_d}{g_i - g_d}$
5. Calculate: $\Delta g_i = g_i - g_d$
6. Check if: $\left|\Delta g_i\right| \le 0.01$
7. If the condition in step 6 is fulfilled, the iteration is completed. We find the required sag corresponding to the unit load $g_h\left(g_f\right)$. In case the condition is not fulfilled, repeat the procedure from step 1 with: $i = i + 1$.

4.4.2.3 Sag Calculation of a Suspension Bridge

In suspension bridges, the geometry of the cable is also parabolic. The elongation of the cable due to the change in stress is considered for the cable part within the tower saddles only. The tower in the LSTB and SSTB standards is a hinged tower. This means that the tower can lean on its hinged bottom to the front, i.e., to the river side, or backward, i.e., to the anchorage side. The saddles have the same levels. The backstay distances are different. They depend on the topography of the slope or bank, and are defined by the backstay cable inclination. The inclination is in the range of 18° -30°.

The relation of cable sag in respect to the change in cable length and in respect to the change in span length due to the displacement of the tower saddles can be expressed as:

$$\Delta b = \frac{15\Delta L}{aa} + \frac{bb\Delta\ell}{aa}$$

Here $\Delta\ell$ is the sum of displacement of both tower saddles due to the change in cable length at the backstays.

$$\Delta\ell = \Delta D_L + \Delta D_R$$

The elastic elongation of the cable at its different parts is expressed as such:

The change in the cable length within the span or between the tower saddles due to a change in cable stress can be calculated as such:

$$\Delta L = L_d \frac{\overline{T}}{EA}\frac{\Delta g}{g} = L_d \frac{(3H + gb)}{3EA}\frac{\Delta g}{g}$$

Here the average tension is assumed as: $\overline{T} = \frac{2H + T}{3} = \frac{2H + H + gb}{3} = H + \frac{gb}{3}$

The displacement of the saddles caused by the change in the cable length of the backstay cables is calculated as such:

$$\Delta D = D\frac{T}{EA}\frac{\Delta g}{g} + \Delta d$$

Here, Δd is the displacement of the saddle caused by the change in the sag of the backstay cable. This displacement affects sag calculation only in the hoisting case.

$$\Delta d_R = \frac{g_h^2 D_R^3}{24\cos\beta_f}\left(\frac{1}{H_d^2} - \frac{1}{H_h^2}\right) \text{ and } \Delta d_L = \frac{g_h^2 D_R^3}{24\cos\beta_f}\left(\frac{1}{H_d^2} - \frac{1}{H_h^2}\right)$$

Displacement of the saddle for the cable hoisting case is:

$$\Delta D_{Rh} = \frac{T_h D_R}{EA}\frac{g_h - g_d}{g_h} + \frac{g_h^2 D_R^3}{24\cos\beta_f}\left(\frac{1}{H_d^2} - \frac{1}{H_h^2}\right)$$

$$\Delta D_{Lh} = \frac{T_h D_L}{EA}\frac{g_h - g_d}{g_h} + \frac{g_h^2 D_L^3}{24\cos\beta_f}\left(\frac{1}{H_d^2} - \frac{1}{H_h^2}\right)$$

For finding the hoisting and full load sags, b_h and b_f, corresponding to the hoisting and full unit loads, g_h and g_f, respectively, the following procedure of iteration is carried out:

0. Assume: $b_* = 0.98b_d$ for hoisting load case and
   $b_* = 1.05b_d$ for full load case.

1. Calculate: $L_d = \ell\left[1+\frac{8}{3}\left(\frac{b_d}{\ell}\right)^2\right]$

2. Calculate: $aa = 16\,\frac{b_d}{\ell}\left[5 - 24\left(\frac{b_d}{\ell}\right)^2\right]$

3. Calculate: $bb = 15 + 40\left(\frac{b_d}{\ell}\right)^2 + 288\left(\frac{b_d}{\ell}\right)^4$

4. Calculate the approximate full load angle: $Tan\beta_{fo} = 4\frac{1.05b_d}{\ell} = \frac{4.2b_d}{\ell l}$

5. Start iteration with: *i = 1, 2, 3 ………*

6. Calculate: $H_i = \frac{g_h(g_f)\ell^2}{8b_i}$ and $T_i = H_i\sqrt{1+16\left(\frac{b_i}{\ell}\right)^2}$

7. Calculate: $\Delta L_i = \frac{(2H_i + T_i).L_d}{3EA}\cdot\frac{g_i - g_d}{g_i}$

8. Calculate: $\Delta D_R^i = \frac{T_i.D_R}{E.A}\cdot\frac{g_i - g_d}{g_i} + \left[\frac{g_h^2.D_R^3}{24\cos\beta_{fo}}\left(\frac{1}{H_d^2} - \frac{1}{H_h^{i\,2}}\right)\right]$

$$\Delta D_L^i = \frac{T_i.D_L}{E.A}\cdot\frac{g_i - g_d}{g_i} + \left[\frac{g_h^2.D_L^3}{24\cos\beta_{fo}}\left(\frac{1}{H_d^2} - \frac{1}{H_h^{i\,2}}\right)\right]^{**}$$

** The term inside the bracket is relevant for the hoisting case only. It has to be omitted in the full load case.

9. Calculate: $\Delta b_i = \frac{15\Delta L_i}{aa} + \frac{bb\Delta D_i}{aa}$

10. The new $\Delta b_{i+1} = b_d + \Delta b_i$

11. Check if $|b_{i+1} - b_i| \le 0.005$

12. If the condition in step 11 is fulfilled, the sag is: $b_h(b_f) = b_i$ and if the condition is not fulfilled, then repeat the procedure from step 5 with *i = i + 1*.

The cable size so far used in trail bridges in Nepal are of diameter, ϕ13 mm, ϕ26 mm, ϕ32 mm, ϕ36 mm and ϕ40 mm. Now in the SSTB standard bridge, only cables of ϕ13 mm, ϕ26 mm and ϕ32 mm diameters are used. In the LSTB design, all the above mentioned sizes are applicable.

In SSTB Suspended type bridges, there are two handrail cables, one on each side of the axis. They are often of ϕ26 mm diameter cables, except in spans above 105 m with 106 cm walkway width, in which ϕ32 mm diameter cables are used for the handrails. Similarly, main cables can be up to four numbers.

In LSTB Suspended type bridges, there are two handrail cables of ϕ26 mm diameter. This size is applicable for bridges of up to 185 m span. For bridges with a span greater than 185 m, the handrail cables are of ϕ 40 mm diameter. Similarly, the main cables can be six numbers for drum type anchorage and 8-12 numbers for open type anchorage.

In the SSTB standard bridge, the cable combination is selected from the table given below according to the span and walkway width. The table of these cable combinations has been made on the basis of sag calculations for every span range given in the table with the condition that the dead load sag is in the ratio: $\frac{Span}{22}$. The level difference between the saddles, h does not exceed the value: $\frac{Span}{25}$. The weight of the walkway excluding the main and hand cables is 40 kg/m span and 55 kg/m span for 70cm and 106cm walkway widths respectively. The live load is as per norms: for span over 50 m, $p = \left(300 + \frac{5000}{span}\right).walkway / width.$ kg/m span and for span up to 50 m, $p = 400$ x walkway width, kg/m span. The cable properties are given above in Chapter 4.2.2.

4.4.3 Cable Design for a Suspended Bridge

4.4.3.1 Short Span Trail Bridge

In the SSTB standard, the cable design is done with predefined cable geometry and as per span and walkway width of the proposed bridge. The cables can be selected from the table bellow.

Walkway 70 cm	Walkway 106 cm	Cable combination		Cable weight, g_h
Max. Span (m)	Max. Span (m)	n x ϕ mm	n x ϕ mm	kg/m
50	40	2 x 26	2 x 26	10.04
90	60	2 x 26	2 x 32	12.62
100	75	2 x 26	4 x 26	15.06
120	105	2 x 26	4 x 32	20.22
-	120	2 x 32	4 x 32	22.80

The hoisting sag, $b_h = 0.95.b_d$ (m), where b_d is the dead load sag = $\frac{Span}{20(22)}$ as per span range. The level of lowest point from the lower saddle is at a vertical distance: $f_{\max}^{hoist} = \frac{(4b_h - h)^2}{16.b_h}$.

The full load sag is:
$b_f = 1.2.b_d$ (the coefficient 1.2 is same for the span to sag ratio = 22)

The maximum tension in the cable in full load case is: $T_{\max} = \dfrac{2.35.g_f.\ell}{1000}$ (ton).

The entrance angle at dead load case is: $\beta_{dead} = 13.5^o - 9.1^o$ for span to sag ratio = 20;
$\beta_{dead} = 12.5^o - 8.1^o$ for span to sag ratio = 22.

The nominal span, ***S*** of a bridge is called the span of the walkway part of the bridge. In the SSTB bridge, it is : $S = \ell - 2x0.6$ (m)

The design procedure is as such:

1. Make a sketch of the profile showing the high flood level and the levels of selected points for cable saddles (higher than the ground level) on both banks.
2. Check the difference of the levels between the saddles, $h \le \dfrac{\ell}{25}$.
3. Check that the level of the higher saddle is higher than the high flood level by **dh**:
   For span, $\ell \le 80.0m \rightarrow dh \approx 5.0 + \dfrac{\ell}{14}$
   and $\ell \succ 80.0m \rightarrow dh \approx 5.0 + \dfrac{\ell}{16}$
4. Once the levels and the span of the saddles have been fixed, check the freeboard again.
   For that, calculate the ordinate of the lowest point: $f_{\max}^{dead} = \dfrac{(4b+h)^2}{16b}$
   Freeboard $= \nabla_{high}^{saddle} - f_{\max}^{dead} - \nabla^{HFL} \ge 5.0m$
5. Select the cable combination from the table according to the chosen walkway width.

4.4.3.2 Long Span Trail Bridge

In the LSTB standard, the cable design is done for the desired dead load sag in the range: $b_d \approx \dfrac{\ell}{23} \rightarrow \dfrac{\ell}{19}$. The sag, $\boldsymbol{b_d}$ and the level difference, ***h*** is so selected that the entrance angle, β_{dead} does not exceed 12°. The conditions are expressed as such:

$$h \le \frac{\ell}{14} \text{ and}$$

$$b_d^{\min} = \frac{l}{23} - \frac{h}{4} \Rightarrow\Rightarrow \beta_d^{\min} \le 10^o$$

$$b_d^{\max} = \frac{l}{19} - \frac{h}{4} \Rightarrow\Rightarrow \beta_d^{rec} \le 12^o$$

The handrail cables and the main cables have the same dead load sag. The handrail cables are always two numbers. The sag calculation is performed for the combination of all the cables as a single unit. The distribution of the tension in the cables is done as such:

$$T = T_{main} + T_{hand}$$

The stress is: $\sigma = \dfrac{T}{A_{main} + A_{hand}} = \dfrac{T_{main}}{A_{main}} = \dfrac{T_{hand}}{A_{hand}}$. And from here we get:

$$T_{main} = T \frac{A_{main}}{A_{main} + A_{hand}} \qquad and \qquad T_{hand} = T \frac{A_{hand}}{A_{main} + A_{hand}}$$

Here, A_{main} and A_{hand} are the metallic areas of the main cables and handrail cables respectively.

The designing of the cable has to be done by following these steps:

1. Make a sketch of the profile and locate the cable saddles on both the banks.
2. Define the design span, l and check the level difference, $h \le \dfrac{\ell}{14}$.
3. Check if the freeboard is sufficient. $\text{FB} = \nabla_h^{saddle} - f_{\max}^{dead} - \nabla^{HFL} \ge 5.0m$. In case the span is greater than 120.0 m, a windguy arrangement shall also be included in the design. In that system, the freeboard is counted from the high flood level to the windguy cable system, which, in usual practice, is placed below the lowest point of the walkway.
4. The approximate maximum tension in the cables is:

$$T_{\max} \approx \frac{g_f \ell^2}{8b_f} Sec12^o = \frac{4.70\ell^2}{8x1.2b_d} 1.022 = \frac{0.5\ell^2}{\frac{\ell}{22}} \approx 11.\ell \text{ kN}$$

   For cables with a maximum tension of upto 2088 kN, the handrail cable size can be ϕ26 mm. If the tension is greater than 2088 kN, the handrail cable shall be of ϕ40 mm diameter as per the anchorage standard design. The number and sizes of the main (Walkway) cables have to be so selected that the permissible tension of the total cables should be as close to the calculated maximum tension as possible.
5. The dead load should include the weight of the windguy and wind-ties cables. A windguy is not necessary for spans of less than 120m. When calculating the sag, the following windguy cable sizes can be used in the initial stag, in a dead load case. For spans of up to 200 m, the windguy cable size can be ϕ26 mm. And for longer spans of up to 300 m, a ϕ32 mm windguy cable should be sufficient.
6. After the sag calculation is completed, the following table shall be filled up:

Load Case	Load, **g** kN/m	Tension, **T**	Sag, **b** m	Lowest point from higher saddle: Hz-dist., **e** m	Lowest point from higher saddle: Ver-dist., ***f*** m	Elevation of lowest point m
Hoisting						
Dead Load						
Full Load						
Live Load		Modulus of elasticity of cable, E kN/mm^2.				

4.4.4 Cable Design for a Suspension Bridge

Like in the suspended bridge, in the SSTB standard suspension bridge, cables of ϕ13mm, ϕ26mm and ϕ32mm diameters are used. While in the LSTB suspension bridge, cables of ϕ13mm, ϕ26mm, ϕ32mm, ϕ36mm and ϕ40mm diameters are applicable.

4.4.4.1 Short Span Trail Bridge

The span of a SSTB suspension bridge has to be adjusted to an integer value. The first suspender from the tower is at a distance 2.5 m for an uneven span, and 3.0 m for an even span. The higher value is for wider tower foundations. The suspender to suspender spacing is 1.0 m. The walkway width is always 106 cm. The walkway (spanning) cable is always of 26 or ϕ32mm diameter. The main cables can be 2xϕ26mm, 4xϕ26mm, 2xϕ32mm and 4xϕ32mm. The tower heights are 5.50, 7.35, 9.20 and 11.05 m. The level difference between the vertexes of the main cable and the walkway cable at mid-span is 1.10 m.

The camber is in general 1.5% of the span. The pretension in the bridge due to the walkway cable is deducted as such:

a. The walkway cable is hoisted and fitted to the walkway cable anchors with the help of a pulling machine. In usual practice, the pulling machine has a maximum capacity of 5.0 tons. Considering this fact, it is assumed that the tension at the ends of the walkway cable is equal to 5.0 tons.

b. The dead load camber is: $c_d = 0.015 x \ell$

c. Then the pretension borne by both the walkway cables is calculated as such:

$$2x5000 = \frac{gp_d \ell^2}{8.c_d}\sqrt{1+\left(\frac{c_d}{\ell}\right)^2} \Rightarrow\Rightarrow gp_d = \frac{1200}{\ell}$$

The weight of the walkway system of the SSTB standard suspension bridge includes:

1.	Steel deck	37.21 kg /m
2.	Walkway beam including pipes	37.00 kg/m
3.	Wire mesh	4.05 kg/m
4.	Handrail & Fixation cables 4ϕ13mm	2.56 kg/m
5.	Suspenders (maximum)	8.30 kg/m
6.	Spanning cable 2ϕ26mm	5.02 kg/m
	Total walkway system weight	94.14 kg/m

The live load is as per norms: for spans of over 50m,

$p = \left(300 + \frac{5000}{span}\right).walkway - width$ (kg/m span) and for spans of up to 50m,

$p = 400$ x walkway-width (kg/m span). The cable properties are given above in Chapter 4.2.2.

In the SSTB suspension bridge, the cable combinations and their geometry in accordance with the span is selected from the table given below:

Span	Tower height	Cables size ϕmm		Dead load sag	Hoisting sag	Camber	Full load angle, β_f
		Main	Walkway				
m	m			m	m	m	degree
30.0	5.50	2ϕ 26mm	2ϕ 26mm	4.20	4.13	0.90	29.83
31.0	5.50	2ϕ 26mm	2ϕ 26mm	4.15	4.07	0.95	28.83
32.0	5.50	2ϕ 26mm	2ϕ 26mm	4.10	4.02	1.00	27.88
33.0	5.50	2ϕ 26mm	2ϕ 26mm	4.10	4.01	1.00	27.24
34.0	5.50	2ϕ 26mm	2ϕ 26mm	4.08	3.99	1.02	26.55
35.0	5.50	2ϕ 26mm	2ϕ 26mm	4.05	3.95	1.05	25.84
36.0	5.50	2ϕ 26mm	2ϕ 26mm	4.22	4.12	0.88	26.14
37.0	5.50	2ϕ 26mm	2ϕ 26mm	4.19	4.09	0.91	25.49
38.0	5.50	2ϕ 26mm	2ϕ 26mm	4.16	4.05	0.94	24.88
39.0	5.50	2ϕ 26mm	2ϕ 26mm	4.13	4.01	0.97	24.31
40.0	5.50	2ϕ 26mm	2ϕ 26mm	4.10	3.97	1.00	23.77
41.0	5.50	2ϕ 26mm	2ϕ 26mm	4.10	3.96	1.00	23.39
42.0	5.50	2ϕ 26mm	2ϕ 26mm	4.10	3.96	1.00	23.03
43.0	5.50	2ϕ 26mm	2ϕ 26mm	4.30	4.16	0.80	23.48
44.0	5.50	2ϕ 26mm	2ϕ 26mm	4.40	4.26	0.70	23.53
45.0	5.50	2ϕ 26mm	2ϕ 26mm	4.40	4.25	0.70	23.20
46.0	5.50	2ϕ 32mm	ϕ 32mm	3.92	3.77	1.18	20.45
47.0	5.50	2ϕ 32mm	ϕ 32mm	3.90	3.74	1.20	20.10
48.0	5.50	2ϕ 32mm	ϕ 32mm	4.05	3.89	1.05	20.39
49.0	5.50	2ϕ 32mm	ϕ 32mm	4.05	3.88	1.05	20.13
50.0	5.50	2ϕ 32mm	ϕ 32mm	4.10	3.93	1.00	20.07
51.0	5.50	2ϕ 32mm	ϕ 32mm	4.10	3.92	1.00	19.83
52.0	5.50	2ϕ 32mm	ϕ 32mm	4.10	3.91	1.00	19.61
53.0	5.50	2ϕ 32mm	ϕ 32mm	4.10	3.90	1.00	19.40
54.0	5.50	2ϕ 32mm	ϕ 32mm	4.10	3.88	1.00	19.20
55.0	5.50	2ϕ 32mm	ϕ 32mm	4.25	4.04	0.85	19.46
56.0	7.35	2ϕ 32mm	ϕ 32mm	5.47	5.30	1.48	22.82
57.0	7.35	2ϕ 32mm	ϕ 32mm	5.44	5.26	1.51	22.46
58.0	7.35	2ϕ 32mm	ϕ 32mm	5.41	5.22	1.54	22.12
59.0	7.35	2ϕ 32mm	ϕ 32mm	5.38	5.18	1.57	21.79
60.0	7.35	2ϕ 32mm	ϕ 32mm	5.35	5.13	1.60	21.48
61.0	7.35	2ϕ 32mm	ϕ 32mm	5.35	5.12	1.60	21.26
62.0	7.35	2ϕ 32mm	ϕ 32mm	5.45	5.22	1.50	21.32
63.0	7.35	2ϕ 32mm	ϕ 32mm	5.65	5.42	1.30	21.65

Span M	Tower height m	Cables size ϕmm Main	Cables size ϕmm Walkway	Dead load sag m	Hoisting sag m	Camber m	Full load angle, β_f degree
64.0	7.35	2ϕ 32mm	2ϕ 32mm	5.85	5.63	1.10	21.97
65.0				5.95	5.72	1.00	22.01
66.0	9.20			7.02	6.81	1.78	24.48
67.0				6.99	6.77	1.81	24.15
68.0				6.96	6.73	1.84	23.83
69.0				6.93	6.69	1.87	23.54
70.0				7.20	6.97	1.60	23.97
71.0				7.40	7.17	1.40	24.22
72.0				7.60	7.37	1.20	24.47
73.0				7.70	7.46	1.10	24.47
74.0				7.65	7.40	1.15	24.14
75.0				7.65	7.40	1.15	23.93
76.0	11.05	4ϕ 26mm		8.57	8.36	2.08	25.38
77.0				8.54	8.33	2.11	25.07
78.0				8.51	8.29	2.14	24.77
79.0				8.48	8.25	2.17	24.48
80.0				8.45	8.22	2.20	24.20
81.0				8.42	8.18	2.23	23.93
82.0				8.39	8.14	2.26	23.67
83.0				8.36	8.10	2.29	23.41
84.0				8.33	8.06	2.32	23.16
85.0				8.30	8.02	2.35	22.92
86.0				8.27	7.98	2.38	22.68
87.0				8.24	7.94	2.41	22.46
88.0				8.21	7.90	2.44	22.24
89.0				8.35	8.04	2.30	22.34
90.0				8.55	8.24	2.10	22.56
91.0				8.85	8.54	1.80	22.96
92.0				9.05	8.74	1.60	23.16
93.0				9.25	8.95	1.40	23.36
94.0				9.25	8.94	1.40	23.20
95.0				9.20	8.87	1.45	22.96
96.0		4ϕ 32mm		7.98	7.68	2.67	20.00
97.0				8.05	7.75	2.60	20.00
98.0				8.13	7.82	2.52	20.00
99.0				8.20	7.89	2.45	20.00

Span	Tower height	Cables size ϕmm		Dead load sag	Hoisting sag	Camber	Full load angle, β_f
		Main	Walkway				
m	m			m	m	m	degree
100.0	11.05	4ϕ 32mm	2ϕ 32mm	8.28	7.96	2.37	20.00
101.0				8.35	8.03	2.30	20.00
102.0				8.40	8.07	2.25	20.00
103.0				8.50	8.17	2.15	20.00
104.0				8.57	8.23	2.08	20.00
105.0				8.65	8.31	2.00	20.00
106.0				8.71	8.36	1.94	20.00
107.0				8.80	8.45	1.85	20.00
108.0				8.85	8.49	1.80	20.00
109.0				8.94	8.58	1.71	20.00
110.0				9.00	8.63	1.65	20.00
111.0				9.00	8.62	1.65	19.88
112.0				8.95	8.55	1.70	19.70
113.0				8.95	8.54	1.70	19.60
114.0				8.95	8.53	1.70	19.50
115.0				8.90	8.46	1.75	19.34
116.0				8.90	8.45	1.75	19.24
117.0				8.85	8.38	1.80	19.09
118.0				8.85	8.37	1.80	19.00
119.0				8.85	8.36	1.80	18.92
120.0				8.85	8.34	1.80	18.83

4.4.4.2 Long Span Trail Bridge

The span of an LSTB standard suspension bridge is adjusted to the value, fitting suspender to suspender spacing of 1.2 m and fixed distances of 2 x 2.30 m of the first suspenders from the towers. The distance from the tower to the first walkway cross beam (without suspender) is 1.1 m.

<u>SPAN</u>

With these conditions, the span is: $\ell = 2.40.i + 2x1.10$ m, where ***i*** is an integer number.

<u>LIVE LOAD</u>

The walkway width is fixed to 1.2 m. The live load is:

For $\ell \leq 50.0m \rightarrow\rightarrow p = 4.80$ kN/m span

For $\ell \succ 50.0m \rightarrow\rightarrow p = 3.60 + \dfrac{60}{\ell}$ kN/m span

CABLE

The LSTB suspension bridge uses these cables:

Cable	Main	Walkway	Windguy	Handrail and Fixation
Applicable quantity, Nos.	2, 4, 6 or 8	2	2	2
Applicable size, ϕ mm	26, 32, 36 or 40	26, 32, 36 or 40	26, 32, 36 or 40	13

TOWER

<table>
<tr><th>S.N.</th><th>Tower height, m</th><th>No. of Main cables, nos.</th><th>Leg to leg distance, c/c₁, m</th><th>Anchor rod to rod distance, c/c₂, mm</th><th>Weight of one tower kg</th></tr>
<tr><td>1</td><td>12.90</td><td>2</td><td rowspan="5">3.5</td><td>383</td><td>1510.93</td></tr>
<tr><td>2</td><td>12.92</td><td rowspan="4">4</td><td rowspan="4">488</td><td>1874.60</td></tr>
<tr><td>3</td><td>14.77</td><td>2119.15</td></tr>
<tr><td>4</td><td>16.62</td><td>3439.08</td></tr>
<tr><td>5</td><td>18.47</td><td>3807.31</td></tr>
<tr><td>6</td><td>17.74</td><td rowspan="5">4 or 6</td><td rowspan="11">4.00</td><td rowspan="3">550</td><td>5210.00</td></tr>
<tr><td>7</td><td>20.24</td><td>5909.48</td></tr>
<tr><td>8</td><td>22.73</td><td>6608.96</td></tr>
<tr><td>9</td><td>25.23</td><td rowspan="8">566</td><td>7861.02</td></tr>
<tr><td>10</td><td>27.73</td><td>9222.19</td></tr>
<tr><td>11</td><td>30.22</td><td rowspan="3">6</td><td>10017.64</td></tr>
<tr><td>12</td><td>30.72</td><td>10823.52</td></tr>
<tr><td>13</td><td>35.21</td><td>12484.87</td></tr>
<tr><td>14</td><td>30.22</td><td rowspan="3">8</td><td>10039.40</td></tr>
<tr><td>15</td><td>30.72</td><td>10845.88</td></tr>
<tr><td>16</td><td>35.21</td><td>12506.63</td></tr>
</table>

Limits and recommendations:

a. Main cable inclination at the saddle in full load and dead load sags

Maximum angle: $\beta_{full} = 30^o \Rightarrow b_d = \dfrac{\ell . Tan 30^o}{1.05x4.0} = 0.137x\ell$

Minimum angle: $\beta_{full} = 20^o \Rightarrow b_d = 0.087x\ell$

Recommended : Select optimum in between 20° to 30°

b. Camber of walkway cable in dead load and tower height

Maximum camber: $c_d = 0.03\ell \Rightarrow h_t = b_d^{\max} + 1.30 + c_d^{\max} - 0.25 = 1.05 + 0.167\ell$

Minimum camber: $c_d = 0.02\ell \Rightarrow h_t = b_d^{\max} + 1.30 + c_d^{\max} - 0.25 = 1.05 + 0.107\ell$

Recommended : Select optimum in between Maximum and Minimum

In the LSTB suspension bridge, the vertical distance between the main and the walkway cables at mid-span is 1.3 m, and the level difference between the tower bottom and the point of intersection made by the walkway cable parabola with the vertical tower axis is 0.25m.

The procedure of designing the cable is as follows:

1. Draw a profile and fix the approximate span. Check the level for sufficient freeboard. And calculate the correct span.
2. Calculate the recommend tower height. Adjust to the nearest standard tower height - one higher and other lower.
3. Calculate one higher and other lower dead load sags: $b_d = h_t - 1.05 - 0.025\ell$
4. Calculate the approximate full load, ${}^{App}g_f = 4.9 + \frac{60}{\ell} + \frac{\ell}{500}$ kN/m span.
5. Calculate the approximate maximum tensions: $T_{\max}(approx.) = \frac{{}^{App}g_f \ell^2}{8.4 b_d}\sqrt{1 + 17.64\left(\frac{bd}{\ell}\right)^2}$
6. Select the number and the sizes of the main cables.
7. Calculate the full load inclination and measure the backstay distances drawing a line from the tower saddle to touch the ground.
8. Assume a size of the windguy cable as per span:
   Windguy cable ϕ26mm for *l* = 120m - 200m,
   Windguy cable ϕ32mm for l > 200m.
9. Calculate the pretension in the walkway cable as for the SSTB suspension bridge with the assumption that one walkway cable is pulled by a 5.0 ton force.

$$2x50 = \frac{gp_d \ell^2}{8.c_d}\sqrt{1 + \left(\frac{c_d}{\ell}\right)^2} \Rightarrow\Rightarrow gp_d = \frac{800.c_d}{\ell}\frac{1}{\sqrt{\ell^2 + c_d^2}} \text{ kN/m}$$

10. Calculate the hoisting, dead and full loads. In the full load case, due to the increase in the main cable sag, the walkway cable will be loosened and will loose its initial pretension. Hence, the full load value excludes pretension. The loads are:

a.	Hoisting load (weight of main cable), g_h	=		kN/m
b.	Dead load, g_d			
	- hoisting load, g_h	=		kN/m
	- steel deck	=	0.47	kN/m
	- walkway support	=	0.27	kN/m
	- handrail/fixation cables	=	0.03	kN/m
	- wire mesh netting	=	0.06	kN/m
	- suspender (average)	=	0.17	kN/m
	- walkway cable weight	=		kN/m
	- windguy cable weight	=		kN/m
	Sub-total dead weights, gd_d	=		kN/m
	- pretension, gp_d	=		kN/m
	Total dead load ($gd_d + gp_d$)	=		kN/m
	Full load, gf			
	- dead weights, gd_d	=		kN/m
	- live load, p	=		kN/m
	Total full load	=		kN/m

11. Calculate the sag by iteration. Correct the backstay distances.

4.5 ELONGATION DUE TO TEMPERATURE EFFECTS

Change in cable length fixed to rigid supports due to change in temperature

$\Delta L = \alpha \, \Delta t \, L$

Here, α Coefficient of thermal expansion ≈ 0.000012 per degree centigrade.
Δt Difference in temperature in degree centigrade.

And in terms of sag to temperature difference:

$$\Delta b = \frac{15}{aa} \alpha \Delta t$$

4.6 WIND EFFECT AND BRIDGE STABILITY

The cable supported trail bridge is not a stiffened structure, hence its movement in any plane, either with live load or alone in dead load condition, is an issue. This issue is important both for the serviceability and for the safety of the bridge. There are many unknown factors influencing the stability of the bridge. The margin of the acceptable movements of such cable supported trail bridges has also not yet been defined. Here, the effect of the wind in bridge stability and in the safety of the structure will be discussed in brief.

4.6.1 Effects of Wind Loads on Bridge

The wind pressure at a certain place is determined mainly by wind velocity. It is also dependent on the air density, atmospheric pressure and air temperature of that place. The wind pressure also increases with the height of the affecting object above the ground or river level, even at the same location. The maximum wind velocity recorded in Nepal is about 35 meters per second (126 km/hr) in the Mustang area. The wind pressure is directly proportional to the quadratic value of the wind velocity. The wind velocity in the vicinity of rivers and lakes is assumed to increase by 18%. The wind pressure on an object located about 15 meters above the ground or river level can be calculated thus:

$$w = \frac{1}{2} C_x \rho v^2 F \sin^2 \varphi$$

Here,
- C_x Aerodynamic coefficient, depending on air density (ρ), on wind velocity (v), on shape, dimension and roughness of the object affected by wind;
- φ Angle between wind direction and wind affecting surface direction, when wind is considered perpendicular to bridge axis, $\varphi = 90°$;
- V Velocity of wind in m/s;
- F Area exposed to wind effect, m^2.

The wind load of 1.0 kN/m span, adopted in the standard design Manual of Suspension Bridge, corresponds to a wind pressure of 1.5 kN/m^2 resulting from a wind velocity of 45m/s (≅160 km/hr).

According to the Swiss standard SIA 160, wind pressure is equal to1.3 kN/m^2 for a wind velocity of 39 m/s (140 km/hr). As stated earlier, using this wind pressure of 1.3 kN/m^2 for the design of the windguy cable of a trail bridge is recommended. Only in exposed places it is recommended to increase the wind pressure up to 1.5 kN/ m^2.

In general, the area of the walkway component of a trail bridge exposed to the wind is estimated to be 0.3 m^2/m-span. Hence, the wind load taken in the standard design manual is very high. For the static calculation, a wind load of 0.5 kN/m span is good for the design. It is only in exposed places that the wind load has to be taken as 0.6 kN/ m span.

4.6.2 Design of Windguy and Windties Cables

A windguy arrangement is required for both suspended and suspension bridges above 120 m span. If the bridge is designed with a sag/span ratio other than the SSTB design (*l*/b = 20, 22), the provision of a windguy arrangement may be required for the purpose of serviceability and stability rather than for safety against wind load.

4.6.2.1 Windguy Arrangement and Design of Windguy Cable Geometry

There are two types of windguy arrangements. They are:

1. Direct windguy arrangement

The direct windguy arrangement is not effective compared to the parabolic one. Hence, this arrangement will not be studied here.

2. Parabolic windguy arrangement

In fact, the parabolic windguy arrangement is a three dimensioned structural system. The load bearing cable (windguy cable) is connected to the bridge walkway by several windties (cable ϕ13mm), placed at pre-defined fixed intervals. The standardized intervals is 5 meters for SSTB type suspended and suspension bridges and 6 meters for LSTB suspended bridges, and 4.8 meters for LSTB suspension bridges. The windguy cable takes a parabolic shape both in the plan and the vertical plane within the part of the bridge span bounded by extreme windties. The self-weight of the cable and its influence in the shape in the vertical plane is omitted in the calculation. In special designs, it is assumed sometimes that the cable is designed to take a straight shape. The outer parts of the windguy cable arrangement, i.e., beyond the first and last wind ties, are assumed to take a straight line shape, both in the plan and vertical planes.

In a suspended type bridge, the windguy arrangement is so designed that the wind guy cable lies below the whole bridge. It gives the bridge additional stiffness to the walkway. Also it supports the bridge to overcome the wind load directed upwards from beneath the bridge.

In a suspension type bridge, it is also usual to put the windguy arrangement below the whole walkway part. The pretension in the walkway of a suspension type bridge is lessened under any loading case. Hence, if the windguy system is put at a level near the walkway level, there is no negative effect.

It is desirable that the vertex of the parabola be placed near the vertex of the walkway. But if the location of the windguy foundation position is not suitable as per the desired layout, the vertex of the windguy cable can be shifted to either side. The span to sag ratio is optimal in the range 8-10.

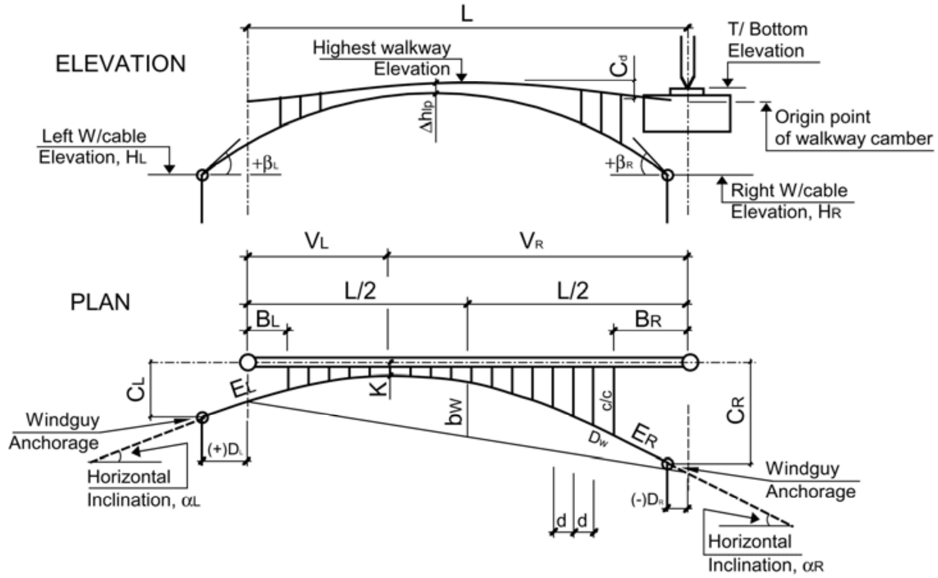
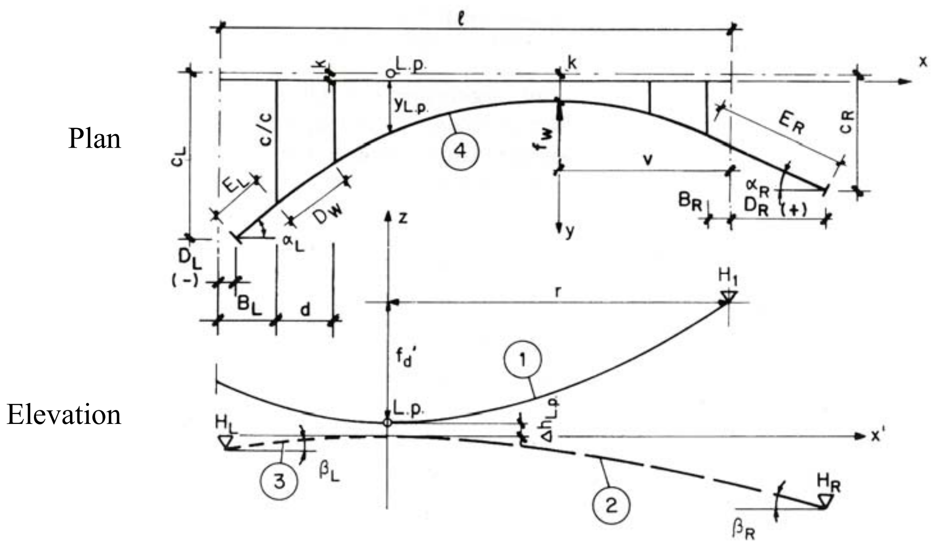


Figure: Windguy arrangement for suspension and suspended types of bridges



Related Symbols:

BL / BR Distance of the first wind-tie along the bridge axis from the left/right main cable saddle in plan (saddle of the main foundation in D-type and tower axis in N-type).

CL / CR Perpendicular distance of the windguy cable anchor from the bridge axis.

CL_0 / CR_0 Perpendicular distances CL / CR at the main cable saddle.

DL / DR Distance of the windguy cable anchor along the bridge axis from the main cable saddle in plan (sign - for the location of the windguy cable anchor inside the span of the bridge and sign + for the location outside the span)

αL / αR Angle between the windguy cable direction and the bridge axis at anchor in plan.

βL / βR Angle of the windguy cable direction with respect to the horizon at anchor in vertical plane.

HL / HR Elevation of windguy cable anchor.

TL / TR Tension in windguy cable anchor.

EL / ER Inclined distance of first wind-tie along the windguy cable from the anchor.

VL / VR	Distance of origin of windguy parabola in plan from the main cable saddle.
eL / eR	Distance of walkway vertex from the saddle in dead load case.
fwL / fwR	Ordinate of windguy parabola in plan at the first wind-tie position.
H1	Elevation of the walkway vertex (in D-type lowest point and in N-type highest point of walkway).
d	Spacing between wind-ties in plan along the bridge axis.
k	Distance to the wind-tie connecting point of the walkway beam from its centre. k = 1.50m for SSTB and 2.20m for LSTB.
sw	Offset distance of axis passing through the origin of the windguy parabola and parallel to the bridge axis in plan.
L	Design span of the bridge.

The formulas reflecting the windguy parabola is as following:

$$fwR = \frac{(CR - sw)(VR - BR)}{VR + BR + 2DR}$$

$$fwL = fwR\left(\frac{L - VR - BL}{VR - BR}\right)^2$$

$$CR_0 = 2\,fwR\frac{BR}{VR - BR} + fwR + sw$$

$$CL_0 = 2\,fwL\frac{BL}{VL - BL} + fwL + sw$$

$$bw = \frac{CR_0 + CL_0}{2} + fwR\left(\frac{0.5L - VR}{VR - BR}\right)^2 - sw$$

$$CL = 2\,fwL\frac{BL + DL}{VL - BL} + fwL + sw$$

If the value of ***bw*** is not in between L/8 – L/10 proceed to choose an appropriate CR, DR and VR so that the result is acceptable.

4.6.2.2 Calculation of Windties

The level of the windguy cable vertex in the vertical plane is dependent of the levels of the windguy anchors on both the banks and the level of the vertex of the bridge walkway.

In standard arrangement, the level of the windguy cable in the vertical plane is defined as such:

CV	Perpendicular distance in plan of a vertical plane passing through both windguy cable anchors from bridge axis at walkway vertex.

∇^V Level of a point in the above plane, defined by the intersection of a line adjoining two windguy anchors and a vertical plane perpendicular to the bridge axis and passing through the vertex of the bridge walkway.

$$CV = CR - \frac{(CR - CL)(eR + DR)}{L + DL + DR}$$

$$\nabla^V = HR - \frac{(HR - HL)(0.5L + DR)}{L + DR + DL}$$

$$\Delta hlp = \frac{H1 - \nabla^V}{CV - k}\left[fwR\left(\frac{eR - VR}{VR - BR}\right)^2 + sw - k \right]$$

$$WV = H1 - \Delta hlp$$

The design of windguy cable is based on the tension developed in the cable for the defined geometry of the windguy cable. The following assumption is considered in the design:

1. The effect of the wind load on the bridge is taken totally by the windguy system. The cooperative resistance of the bridge against wind load is neglected.
2. The increase in the sag of the windguy cable due to wind load is neglected in calculating the tension.
3. The inclined position of the wind guy cable is neglected. The calculation of the tension in the windguy cable is done for wind loads directed perpendicular to the bridge axis and for its layout in plan.
4. The self-weights of windguy and wind-tie cables are omitted.

Windguy cable tension

$$H_W = \frac{wVR^2}{2CR_0}$$

Elevation of windguy cable at windtie positions
For i

$$n = \frac{L - BR - BL}{d} + 1$$

$$m = \frac{VR - BR}{d} + 1$$

$$W_{i=1-m} = WV + \frac{(VR - BR)(WV - HR)}{VR + BR + 2DR}\left[\frac{VR - BR - d(i-1)}{VR - BR}\right]^2$$

$$W_{i=m+1-n} = WV + \frac{(VL - BL)(WV - HL)}{VL + BL + 2DL}\left[\frac{d(i-m)}{VL - BL}\right]^2$$

$$B_i = H1 + fd_R\left[\frac{VR - BR - d(i-1)}{VR - BR}\right]^2$$

$$\Delta h_i = B_i - W_i$$

$$y_i = fR\left[\frac{VR - BR - d(i-1)}{VR - BR}\right]^2 + sw - k$$

$$c/c_i = \sqrt{(y_i^2) + \Delta h_i^2}$$

$$Dw_{i=1-(n-1)} = \sqrt{(y_i - y_{i+1})^2 + d^2 + (B_{i+1} - B_i)^2}$$

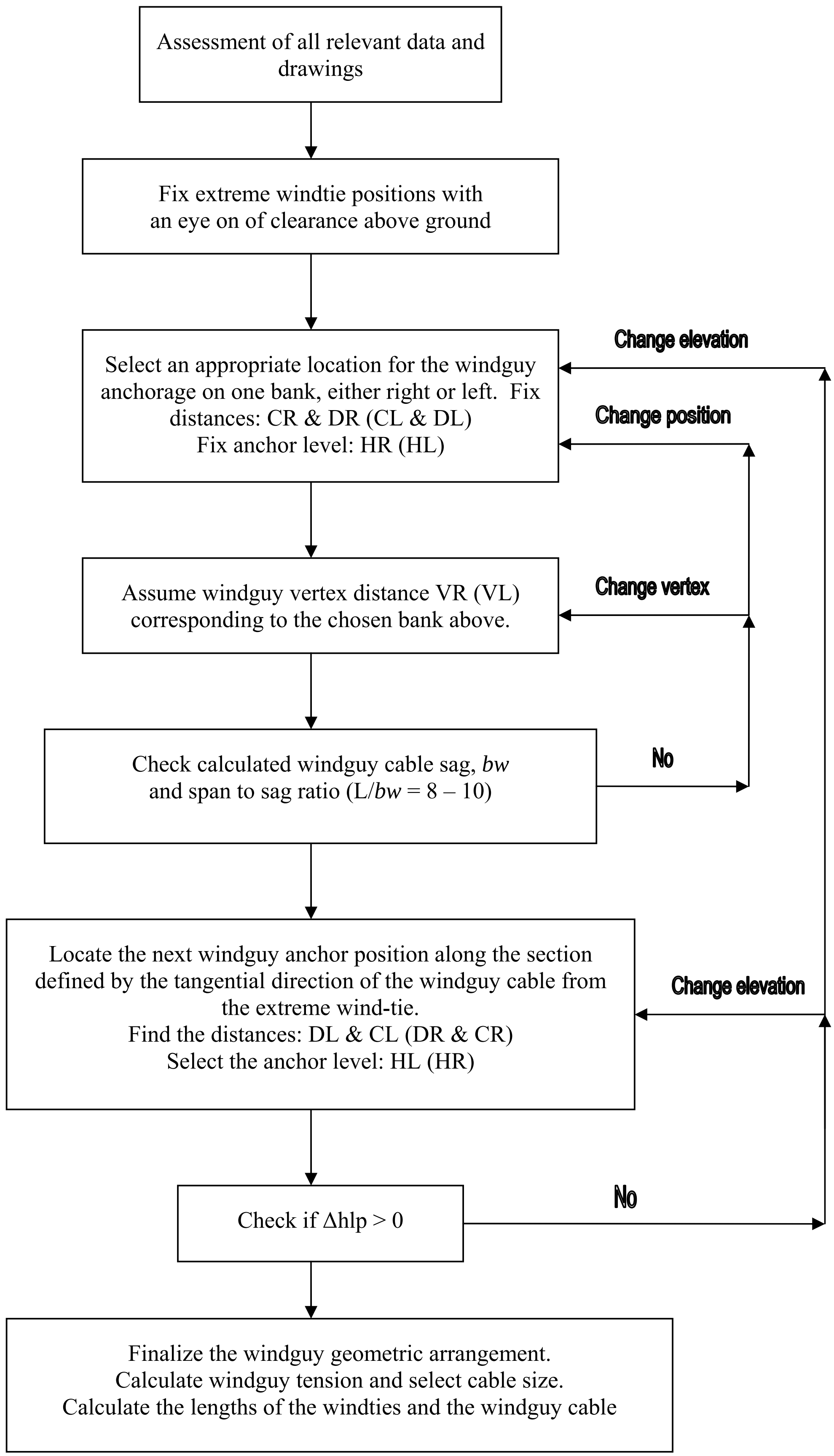
$$ER = \frac{BR + DR}{\cos\beta_R \cos\alpha_R}$$

$$EL = \frac{BL + DL}{\cos\beta_L \cos\alpha_L}$$

$$\sum_i^n c/c_i + 0.8n$$

$$\sum_i^{n-1} Dw_1 + ER + EL + Overlaps$$

$$\sum_i^n c/c_i + 2.2n$$


Assessment of all relevant data and drawings
Fix extreme windtie positions with an eye on of clearance above ground
Select an appropriate location for the windguy anchorage on one bank, either right or left. Fix distances: CR & DR (CL & DL)
Fix anchor level: HR (HL)
Change elevation
Change position
Assume windguy vertex distance VR (VL) corresponding to the chosen bank above.
Change vertex
Check calculated windguy cable sag, bw and span to sag ratio (L/bw = 8 – 10)
No
Locate the next windguy anchor position along the section defined by the tangential direction of the windguy cable from the extreme wind-tie.
Find the distances: DL & CL (DR & CR)
Select the anchor level: HL (HR)
Change elevation
Check if Δhlp > 0
No
Finalize the windguy geometric arrangement.
Calculate windguy tension and select cable size.
Calculate the lengths of the windties and the windguy cable

Design procedure:

1. Lay down your general arrangement drawing and define the extreme positions of the wind-ties. Make sure that they lie at the walkway beams and that their distances from the walkway vertex are multiples of the integral number and interval between wind-ties, **d**. These extreme wind-ties are not disturbed by the slope and there must be sufficient gap below them to the ground to free them from debris cover.

2. Decide on any one bank to start doing the design. The wind-ties will be counted from this bank towards the other. Select on the plan the suitable position for the windguy anchor point. The anchor point is the theoretical end of the windguy cable. Measure the distances: **DR (DL)** and **CR (CL)**, corresponding to the selected first bank. Propose the elevation of the anchor point as per the topographical condition **HR (HL)**.

3. Select the vertex distance of the windguy cable: **VR (VL)**, corresponding to the selected first bank. Make sure that the distance between the windguy vertex and the walkway vertex is a multiple of the integral number and the interval between wind-ties, **d**. Try to arrange the windguy vertex nearest to the walkway vertex.

4. Calculate the sag, ***bw***. Check if the span sag ratio is in the range 8 – 10. If the ratio is not within the range, change the vertex distance **VR (VL)**.

5. Draw a straight line from the point $\mathbf{CL_0}$ $\mathbf{(CR_0)}$, at the angle $\boldsymbol{\alpha}$**L** ($\boldsymbol{\alpha}$**R)** to the bridge axis, calculated earlier for defining the sag, *bw*. Locate an appropriate position for the windguy anchor point. Find the distances: **CL (CR)** and **DL (DR)**. Propose the elevation of the anchor point as per the topographic condition **HL (HR)**.

6. Calculate the value of **Δhlp**. Check if **Δhlp > 0**. If not, change the proposed elevations of the anchor points on both the banks as far as possible to lay the anchor foundations. Initially, try to locate the anchor points on the lines defined earlier. Only as the next alternate, select a new location for the windguy anchor point **CR (CL)** and **DR (DL)** and repeat the whole process from the beginning.

7. Once the geometric arrangement is fixed, find the tensions at the cable ends, and select the size of the cable. In normal designs, cables of ϕ26 mm diameter is good for bridge spans up to 225 m.

8. Calculate the elevations of the walkway and the windguy at the positions of the wind-ties from the first bank to the opposite bank.

9. Calculate: the c/c distances of the wind-ties and the distances Dw along the windguy cable from one to another wind-tie. Calculate also: the distances, **EL** and **ER**; angles **βR**, **βL** and the total lengths of the wind-ties and the windguy cable including overlap.

4.7 DESIGN OF TOWER FOR A SUSPENSION BRIDGE

The towers of trail suspension bridges are, in general, of three types:

i) One is the tower used in suspension type Trail Bridges at present day. These towers are usually made of steel. They have a hinged system at their base. The main load bearing cables rest on the saddle at the top of the tower and are fixed to the tower itself. They cannot slide or roll over the saddle. Changes in the main cable sag due to different loading cases cause the tower to tilt. The leaning of the tower towards the bridge side results in a slight change in the span without deforming the tower structure. The tower does not resist change in the cable geometry that balances the relation between the sag and the span.

ii) The second is the tower of the Scottish type suspension bridge made before 1950 or the concrete tower of local type suspension bridges. They have a fixed base and the main cables are clamped and secured to the tower tops without possibility of the main cables sliding or rolling over the tower top (saddle). The span of the bridge is fixed. The structures of such towers act as a vertical cantilever.

iii) The third is the masonry and reinforced concrete or steel tower with a fixed base and saddle on the top, which allows the main cables to slide or roll over it. The towers or so-called handrail posts of local, SSTB and LSTB type suspended bridges have a fixed base, and theoretically, the cables can slide over the saddle. The structure is designed to resist vertical components of the tension in the cable.

4.7.1 Basic Principle of Tower Design

The towers used in Trail Bridges are subject to a number of loading actions. The principal one is, of course, the vertical component of the cable force at the tower top. Added to this, there will also be wind loads, both direct and indirect, via the cables and walkway deck. The horizontal component of the cable force at the tower top tends to move it in a span-wise direction; and so bends the tower like a cantilever in case of a fixed base. And in a tower with a hinged base, this horizontal force pulls the tower forward or backward along the bridge axis on the hinged point.

The towers of standard suspension bridges have a uniform section throughout its height. The stiffness of the tower is such that in case the tower top is free, the tower would become unstable under the vertical load V applied by the cables.

For towers of uniform section, this would occur when

$$V = \frac{\pi^2 EI}{4.h^2} \qquad (1)$$

Here, E.I is the flexural stiffness of the tower and h is its height. Under these conditions, the tower would offer no horizontal resistance to the cables. If we can, at the same time, assume that the cables effectively prevent any further horizontal movement beyond that natural to them (as though supported on rollers on a rigid tower) the tower itself will, so far as the load V is concerned, be like a column fixed at its root and held by a hinge at its top. The critical Euler load for such a column is just under

$$P_E = \frac{2\pi^2 EI}{h^2} \qquad (2)$$

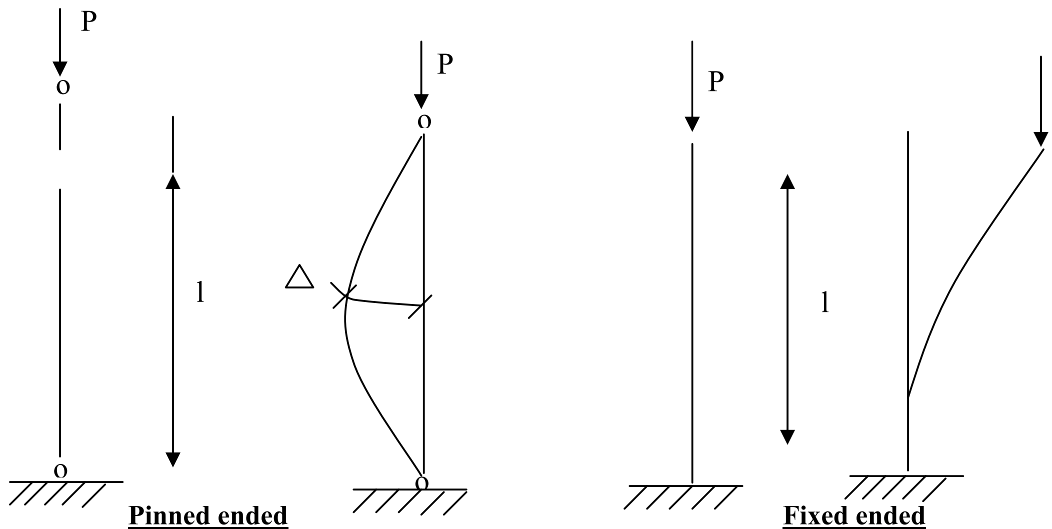
The load factor against instability of the tower at the load V would thus be from (1) and (2)

$$\frac{P_E}{V} = 8 \qquad (3)$$

These figures, of course, relate to an idealized uniform tower but they indicate the practicability of highly flexible towers to be used for suspension bridges.

The critical load necessary for elastic buckling of a bar (tower) can be calculated by using differential equation of the deflection curve as shown in the figure given below where the equation of the critical loading is called Euler's Column Formula. It has to be noted that the critical load is found by determining the value of the axial force, which can cause large deflections even when the lateral itself is very small.

Having established the stability of the flexible tower, we can outline the necessary calculations for its strength with or without wind loading.



Pinned ended

$$P_{\alpha} = \frac{4\pi^2 EI}{l^2}$$

Fixed ended

$$P_{cr} = \frac{\pi^2 EI}{4l^2}$$

4.7.2 Wind Load on Tower Capacity

The global wind load has been accepted to be w = 0.5 kN/m. The effect of a possible vertical load component has not been considered relevant for the design, and therefore is disregarded in the standard design.

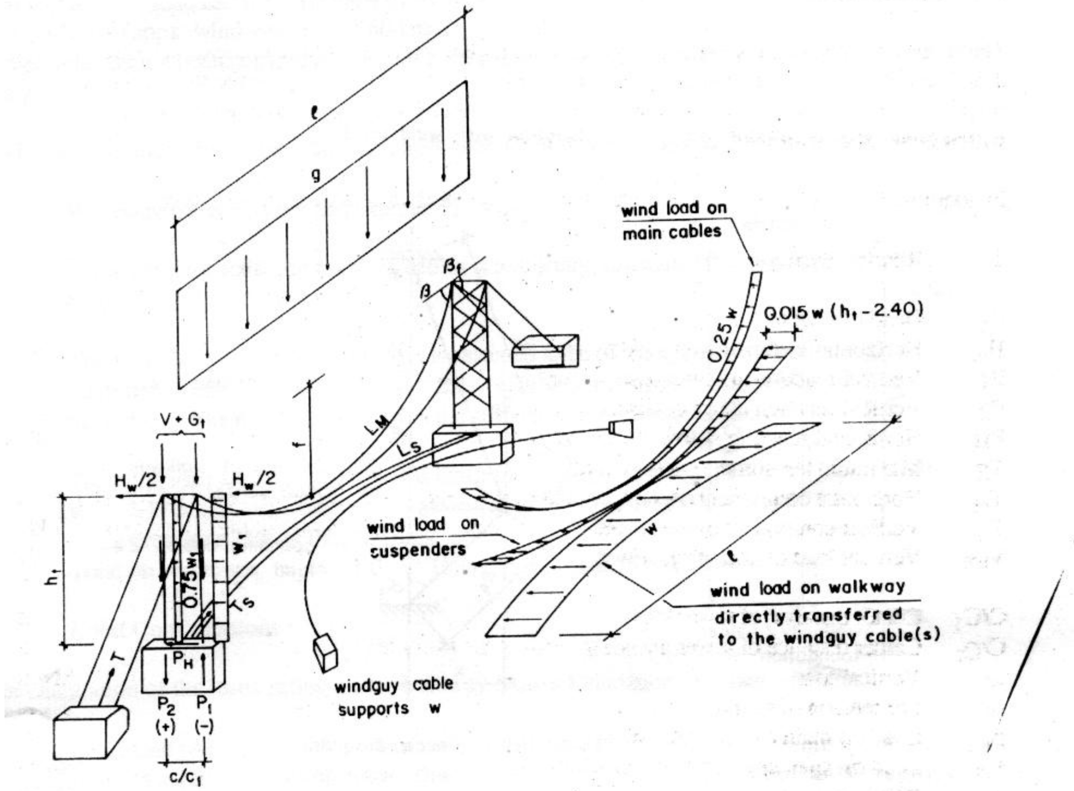
Different elements receive the following direct wind loads perpendicular to the bridge axis:

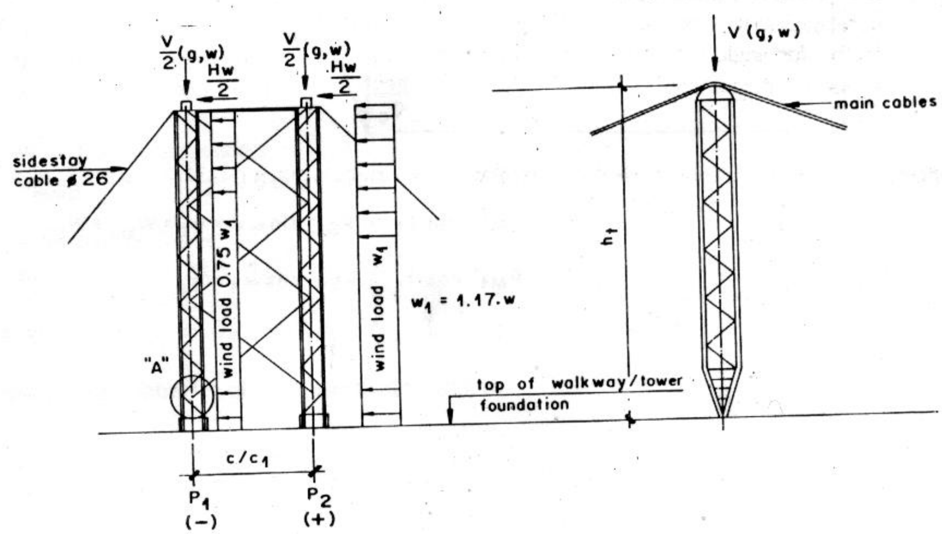
- Walkway, Www=0.5w kN/m
- The wind load acting on the main cables is uniformly distributed and assumed to be: $W_t = 0.25.w$ kN/m
- The wind load acting on suspenders is assumed to be of triangular distribution with a maximum load of: $Ws = 0.015(h_t - 2.4).\ w$ (kN/m)

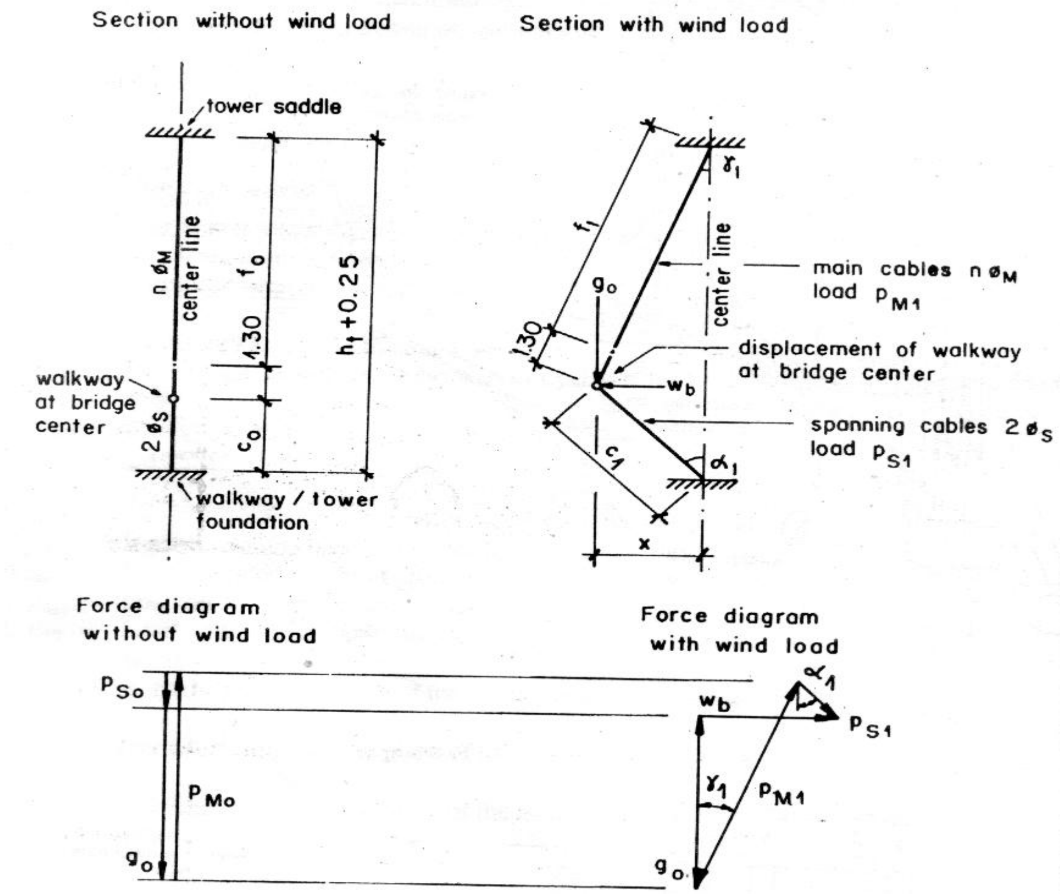
  Acting at the suspender near the towers. The wind load acting
- on the suspender at mid-span is assumed to be zero.

- The wind load acting on the tower is uniformly distributed along the tower height w_1 =1017 w (kN/m), however, the tower leg in front receives 100% and the leg behind receives 75% of the load only.

Total on one tower, $w_t = (1.0 + 0.75)\ w_1 = (1.0 + 0.75).\ 1.17\ w = 2.05\ w$ kN/m







$O = w_b$

$P_{Mo} - P_{So} = g_o$

$P_{M1} \sin \gamma_1 + P_{S1} \sin \alpha_1 = w_b = W_{Mo} + W_{So}$

$P_{M1} \cos \gamma_1 - P_{S1} \cos \alpha_1 = g_o$

4.7.3 Calculation of Loads and Their Combinations

In order to calculate the loads acting on the walkway/tower foundation, two load combinations are considered.

Load case [A] : Dead load case + full wind load
Load case [B] : Full load case + 1/3 wind load

Load case,[A]: w = 1.0 (applicable to CASE (A) kN/m

- Vertical load (dead load case), g_d (including pre-tension of the spanning cable, refer to 8.3)

- Wind load on main cable :

  $W_M = 0.25.1.0 = 0.25$ kN/m

- Wind load on suspenders:

  $$W_{susp} = \tfrac{1}{2}.0.015(h_t - 2.4).1.0 = 0.0075(h_t - 2.4)$$ kN/m

- Wing load on tower:

  Wt = 2.05.1.0 = 2.05 kN/m

  $W_b = W_M + W_{susp} = 0.25 + 0.0075\ (h_t - 2.4) = 0.232 + 0.0075h_t$ kN/m

For case (B) One third of the above loading applies.

Geometrical Parameters

A) General

It is assumed that the main cable and the spanning cable are moving laterally and each will thereby remain in plane. The cable sag (fo) is increased by Df and the camber (Co) by Dc. The cable forces must fulfill the static equilibrium together with the applied loads go, Pso and wbo.

B) Basic Calculation Principle

The different sag (f_1) and camber (C_1) are determined by iteration and the corresponding cable forces are calculated with an assumed geometrical alignment. The sum of the vertical and horizontal components is then compared with the actual vertical load (g_0 + Pso) and the actual horizontal load (w). The difference should be judged, and, if necessary, a new assumption must be made and the calculation repeated until sufficient accuracy is achieved.

Basic Formulas for Iteration

The difference in the horizontal cable tension can be calculated as follows (e.g., for the main cables):

1) $$\Delta H_1 = H_0 - H_1 = \frac{g_1.l^2}{8f_1} - \frac{g_0.l^2}{8f_0}$$ kN

or out of = $$\Delta\sigma_1 = \frac{\Delta H_1}{A} = \frac{\Delta L_1.E}{L_d}$$ kN/mm^2

2) $$\Delta H_1 = \frac{\Delta L_1.E.A}{L_d}$$ kN

Out of the two equations 1) and 2) the load can be calculated as:

3) $$g_1 = \frac{8 f_1.\Delta L_1.E.A}{l^2.L_0} + \frac{f_1}{f_0}.g_0$$ kN/m

Insert into 3) for the cable length difference:

$$\Delta L_1 = L_1 - L_0 = \frac{8}{3l}\left(f_1^2 - f_0^2\right)$$ m

Then g_1 becomes:

4) $$g_1 = \frac{64E.A}{3l^3.L_0}.f_1.f\left(f_1^2 - f_0^2\right) + \frac{f_1}{f_0}.g_0$$ kN/m

$$\frac{64E.A}{3l^3.L_0} = C_0$$ remains constant kN/m^4

1. Calculation Procedure

A) Compile the Initial Layout Data

B) Iteration Procedure

Index 1 means load case 1 either [A] or [B].
Calculate displacement x_1 and sag f_1 by the iteration method for both loading cases.
Draw a force diagram in order to check the results.

The iteration may be started with the following primary values of x and f_1 :

-	Load case (A) :	x_1	=	appx.1.015.l	m
		f_1	=	appx.1.002.f_d	m
-	Load case (B):	x_1	=	appx.0.0025.l	m
		f_1	=	appx.1.001.f_f	m

Step Operation

1. Calculate the constant factors C:

$$C_{Mo} = \frac{64E.A_M}{3l^3 L_{Mo}}$$ kN/m ^{4}

$$C_{So} = \frac{64E.A_S}{3l^3.L_{So}}$$ kN/m ^{4}

It is important that the total metallic area, A_s should include the partial area of steel decking and area of fixation cables. The walkway decking also has to extend when the walkway (spanning) cables is extended.

2. Calculate:

$$\gamma_1 = \arcsin \frac{X_1}{f_1 + 1.30} \quad \text{deg}$$

$$a = \arctan \frac{X_1}{h_t + 0.25 - \cos\gamma_1 .(f_1 + 1.30)} \quad \text{deg}$$

$$C = \frac{h_t + 0.25 - \cos\gamma_1 .(f_1 + 1.30)}{\cos\alpha_1} \quad \text{m}$$

3. Calculate the load on the main and spanning cables:

$$P_{M1} = C_{Mo} . f_1 .\left(f_1^2 - f_0^1\right) + \frac{f_1}{f_0} . P_{Mo} \text{ kN/m}$$

$$p_{S1} = C_{So} . C_1 .\left(C_1^2 - C_o^2\right) + \frac{C_1}{C_o} . P_{So} \quad \text{kN/m}$$

4. Calculate the new f $_1$ and the new x:

new f $_1$ = $f_o + \Delta f .\left[\frac{\Delta P_M - \Delta g}{\Delta P_M}\right]$ m

with: Δf = $f_1 - f_o$ m

ΔP_M = $P_{M1} - P_{Mo}$ kN/m

Δg = $\sum P_{vertical} - g_o = \left(P_{M1} . \cos\gamma_1 - P_{S1} . \cos\alpha_1\right) - g_o$ kN/m

new x_1 = $x_1 . \frac{wb}{\sum P_{horizontal}} = x_1 . \frac{wb}{\left(P_{M1} . \sin\gamma_1 + P_{S1} . \sin\alpha_1\right)}$ m

(Note: if the displacement is too large or iteration is indefinite, tune the value of the displacement, $x_1$or $x_1 = x_1$(old) + 0.01

5. Test the condition: I Δg I < 0.02

- If no (greater): repeat the calculations from step 2 with new f $_1$ and new x $_1$
- If yes (smaller): stop the iteration and proceed with the calculation of the other load cases, complete the force diagram, and then calculate the final data.

C) Calculate the final Data for Load Case[A] and [B]

1. Loads on tower top

Total vertical load $V_{tot} = \frac{P_{M1}.l}{2}.\cos\gamma_1.\left[1 + \frac{l.\tan\beta_f}{4f_1.\cos\gamma_1}\right]$ kN

Horizontal load $H_w = \frac{P_{M1}.l}{2}.\sin\gamma_1$ kN

2. Reactions at the tower base = loads on walkway / tower foundation

$$P_1 = \frac{V_{tot}}{2} + \frac{G_1}{2} - \frac{H_w h_t}{c/c_1} - \frac{1.025w.h_{t^2}}{}$$ kN

$$P_2 = \frac{V_{tot}}{2} + \frac{G_1}{2} - \frac{H_w h_t}{c/c_1} - \frac{1.025w.h_{t^2}}{c/c_1}$$ kN

$$P_H = H_w + 2.05w.h_1 + \frac{P_{S1}.l}{2}.\sin\alpha_1$$ kN

Dead weight of tower:
(for exact weight of towers, see table in Chapter 4.4.4.2)

3. Tension in spanning cables

$$T_{SV} = \frac{P_{S1}.l}{2}.\cos\alpha_1$$ kN

$$T_{SH} = \frac{P_{S1}.l^2}{8_{C1}}$$ kN

Maximum tension in spanning cables (both cables):

$$T_S = \frac{P_{S1}.l^2}{8_{C1}}\sqrt{1 + 16\left(\frac{c_1}{l}\right)^2}$$ kN

Check the safety factor of spanning cable:

$$F_s = \frac{T_{Sbreal}}{T_{SA/B}} \geq 3$$ /

D) Checking the Results
Action on tower base and spanning cable = reaction on walkway / tower foundation = 0

Horizontal (perpendicular to the bridge):

$$\sum H = \tfrac{1}{2} l.w_b + 2.05h_t.w - P_H = 0$$

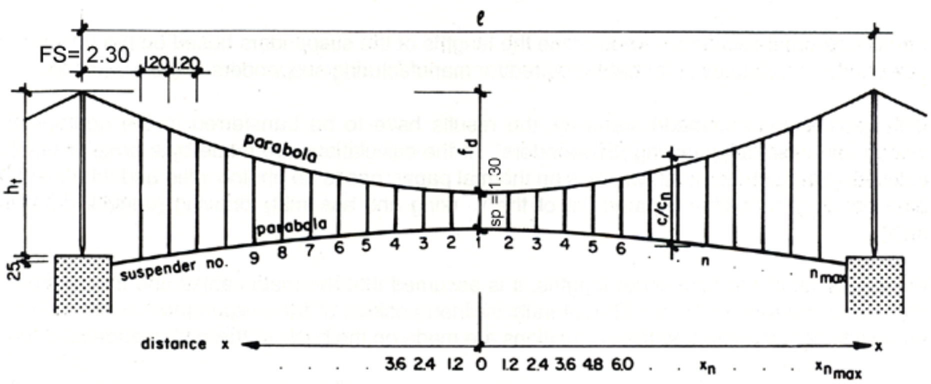
vertical:

$$\sum V = \tfrac{1}{2} l.g_b + G_t + \tfrac{1}{2} V_{tot} - (P_1 + P_2 - T_{SV}) = 0$$

4.8 DESIGN OF SUSPENDERS (BASIC PRINCIPLE)

4.8.1 Basics of Calculation

Layout and Section



Center to center distance of main and spanning cables,

Equation of cable geometry

for main cable: $y_t^M = \frac{4 f_d}{l^2} x_1^2$ for spanning cable: $y_t^S = \frac{4c_d}{l^2} x_1^2$

Here x_i horizontal distance from mid-span to i^{th} suspender
sp the vertical distance between main and spanning cables at mid span
sp = 1.30m in LSTB Standard
=1.10m in SSTB Standard

In the SSTB standard, the spacing between the suspenders is 1.0 m while in the LSTB standard it is 1.2 m.

Hence, $x_i = i$ x1 or i x1.2 m

and $i = 0$ to $\frac{0.5l - FS}{sp}$

In LSTB standard
FS =2.3m

In SSTB standard
FS = 3.0 for even-number span
FS = 2.5 for odd-number span

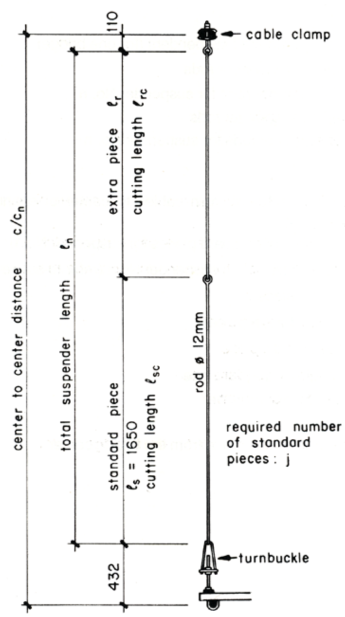
4.8.2 Design of Suspender

The design of the suspender is different for the SSTB and LSTB standards. In the SSTB design, the spacing of the suspender has been fixed as 1m in order to keep the steel deck of the SSTB suspended type bridge fit also for the suspension type. On the basis of practical experience, several modifications have been introduced in the SSTB suspender design.

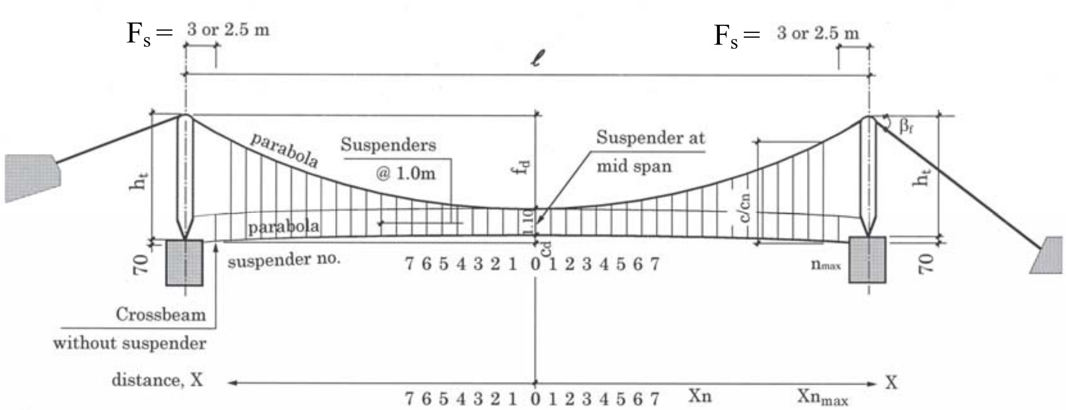
Total suspender length: $l_n = c/c - hd$

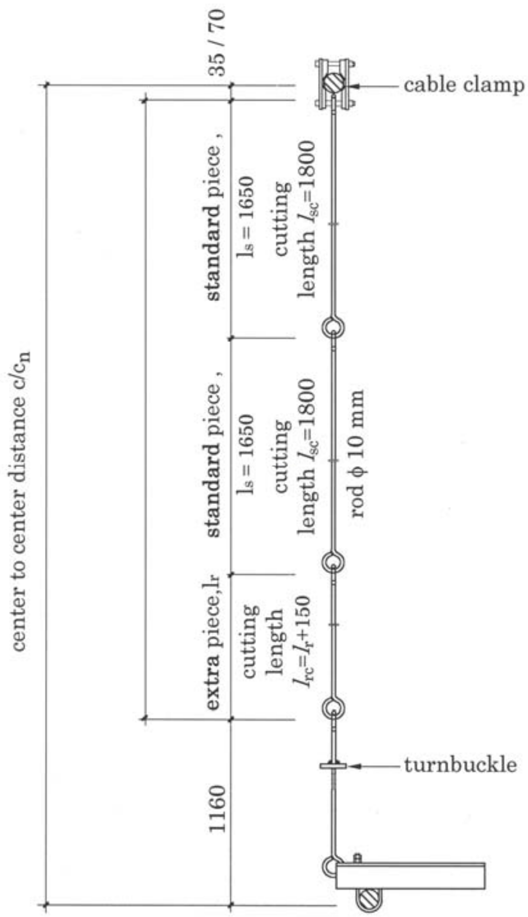
Here *hd* is the distance covered by the walkway hanger rod, turnbuckle and cable clamp. The suspender is composed of a standard piece of definite length and an extra piece of variable length.

Typical Design of Suspender for LSTB Standard



Typical Design of Suspender for SSTB Standard



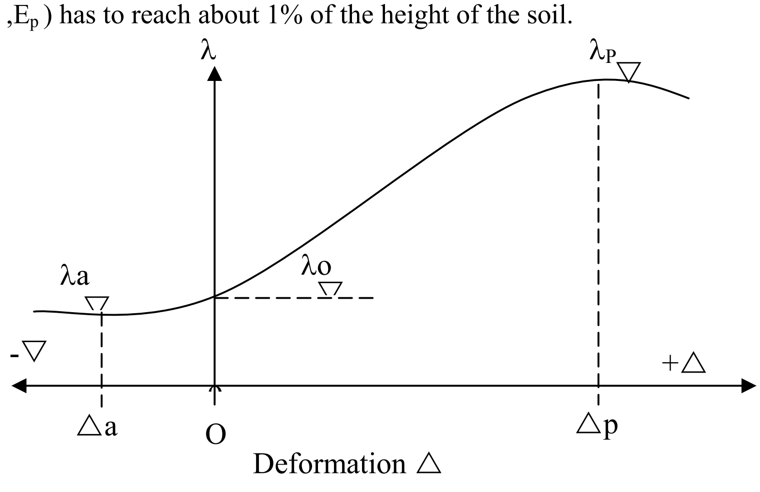


4.9 FUNDAMENTALS OF BRIDGE FOUNDATION

4.9.1 Earth Pressure

Any bridge foundation or retaining structure buried in the soil will have to bear the loads of respective pressures acting from the soil. Depending upon the direction in which the foundation will be moved, these loads are called "active" (movement away from the soil) or "passive" (movement towards the soil) earth pressure loads. If there is no movement, the load is called "earth pressure at rest" (Eo).

In order to develop active earth pressure (e_a,E_a), the necessary movement of the foundation is small, about 0.1% of the supported height. Whereas the deformation of the soil needed to develop passive earth pressure (e_p ,E_p) has to reach about 1% of the height of the soil.



Active Earth Pressure

General case: Active earth pressure per running meter of retaining structure:

$$e_{ah} = \lambda_{ah}.h_a.\gamma \qquad \text{kN/m}^2$$

$$E_{ah} = \frac{1}{2}e_{ah}.h_a = \frac{1}{2}\lambda_{ah}.h_a^2.\gamma \qquad \text{kN/m}$$

$$E_{av} = E_{ah}.\tan(\delta - K) \qquad \text{kN/m}$$

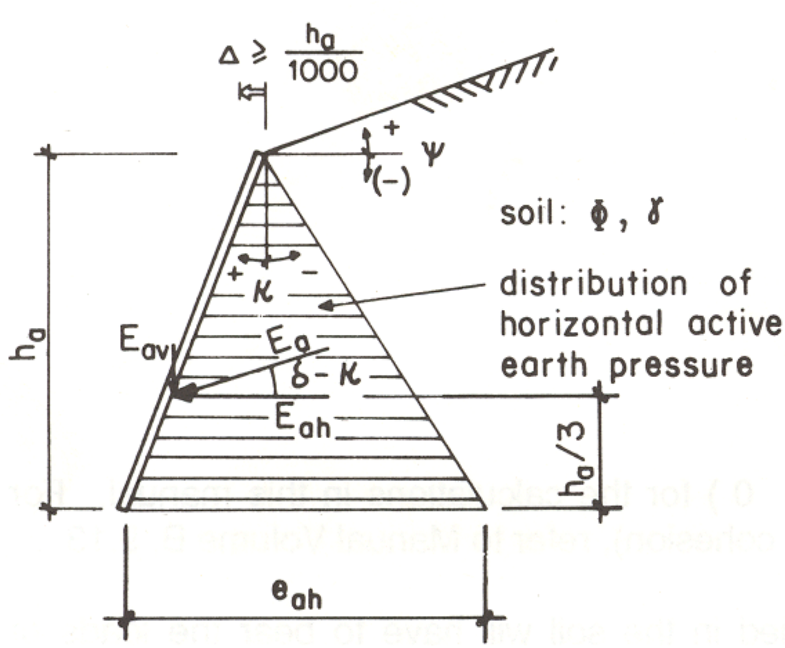
$$E_a = \frac{E_{ah}}{\cos(\delta - K)} \qquad \text{kN/m} \qquad \delta = \frac{2}{3}\Phi$$

$$E_a = \frac{\cos^2(\Phi + K)}{\cos^2 K.\left[1 + \sqrt{\frac{\sin(\Phi+\delta).\sin(\Phi+\Psi)}{\cos(\delta+K).\cos(\psi+K)}}\right]}2$$

and when K = 0,

$$\lambda_{ah} = \frac{\cos^2 \Phi}{\cos^2 K.\left[1 + \sqrt{\frac{\sin(\Phi+\delta).\sin(\Phi+\Psi)}{\cos\delta ... \cos\Psi}}\right]}2$$

(all angles in degree)



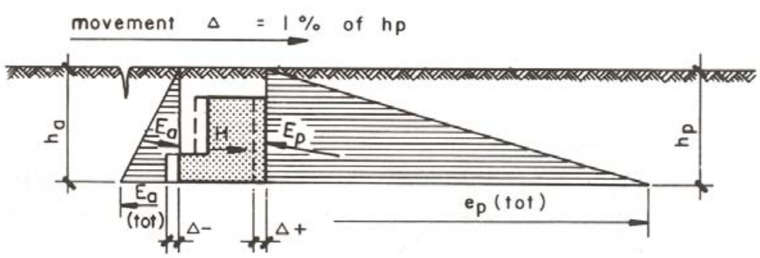
The value of λ_{ah} in relation to Φ and K *for* (and) $\left(\delta = \frac{2}{3}\Phi and \Psi \leq \Phi\right)$

K	Ψ	Φ 25°	27.5°	30°	32.5°	35°	37.5°	40°	42.5°
10°	40°	/	/	/	/	/	/	0.43	0.24
	30°	/	/	0.60	0.30	0.30	0.24	0.20	1.16
	20°	0.44	0.37	0.31	0.26	0.23	0.19	0.16	0.14
	10°	0.34	0.30	0.26	0.22	0.19	0.17	0.14	0.12
	≤°	0.29	0.26	0.23	0.20	0.18	0.15	0.13	0.11
0°	40°	/	/	/	/	/	/	0.59	0.35
	30°	/	/	0.75	0.49	0.39	0.33	0.28	0.24
	20°	0.52	0.45	0.39	0.34	0.30	0.26	0.23	0.20
	10°	0.40	0.36	0.32	0.28	0.25	0.22	0.20	0.17
	≤°	0.35	0.31	0.28	0.25	0.22	0.20	0.18	0.26

Passive Earth Pressure

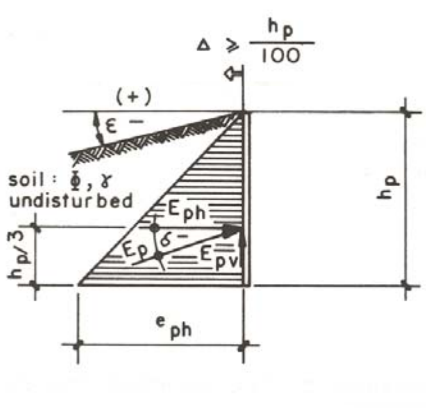
Earth resistance in front of the foundation is not recommended for common bridge foundations except for :

- deadman foundations where the calculations are based on that resistance, and
- for walkway/tower foundations where the earth resistance is taken partially into consideration.



Ea and Ep fully developed

SPECIAL CASE : K = 0



$e_{ph} = \lambda_{ph} \cdot h_p \cdot \gamma$ kN/m²

$E_{ph} = \frac{1}{2} e_{ph} \cdot h_p = \frac{1}{2} \lambda_{ph} \cdot h_p^2 \cdot \gamma \cdot L$ kN/m

$E_{pv} = E_{ph} \cdot \tan\delta$ kN/m

$\delta = -\frac{1}{2}\Phi$

$|\varepsilon| \leq \frac{1}{2}\Phi$

if $\varepsilon > 0$ take $\varepsilon = 0$

$$\lambda_{ph} = \frac{\cos^2\Phi}{\left[1 - \sqrt{\frac{\sin(\Phi - \delta)\cdot\sin(\Phi + \varepsilon)}{\cos\delta \cdot \cos\varepsilon}}\right]^2}$$

The value of λ_{ph} in relation Φ and K

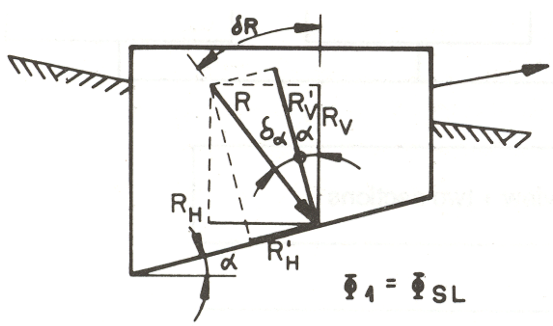
K	ε	Φ							
		25°	27.5°	30°	32.5°	35°	37.5°	40°	42.5°
0°	≥°	3.47	4.06	4.81	5.76	7.02	8.71	11.06	14.45
	-5°	2.85	3.29	3.83	4.50	5.36	6.47	7.96	10.00
	-10°	2.32	2.65	3.05	3.54	4.14	4.90	5.88	7.17
	-15°	1.86	2.11	2.42	2.78	3.21	3.74	4.40	5.25
	-20°	1.42	1.64	1.88	2.15	2.47	2.85	3.31	3.89

4.9.2 Safety Against Sliding

The bridge foundations are the main cable anchorages, windguy cable anchorages and tower/walkway anchorages (suspension bridge). The other accessory structures are retaining walls, drainage and other protective structures. These foundations are the parts of the bridge structure which serve to transmit the load on the structures onto the natural ground.

The major load leading to failure of cable anchorage foundations is the horizontal component of the cable tension which makes the foundation slide forward. When an increase in the driving load results in a shear load which exceeds the shear resistance in the foundation base, a flat foundation being loaded by a more or less horizontal load will start sliding on the subsoil. Shear resistance develops basically due to frictional resistance and interlocking and adhesion between different particles. In our practice, shear force is the product of the normal force acting on the foundation base and the friction angle between the foundation base and the subsoil (or rock). The adhesive characteristic between the foundation base and the subsoil is neglected. Generally, a safety factor of is required against sliding failure. The surface of movement for this failure mode is equal to the contact area between the foundation base and the subsoil (or rock).

The formula for calculating the safety factor against sliding is:



$$F_{SL} = \frac{\text{Re}\,tainingForces}{DrivingForces} = \frac{N.Tan\Phi_{SL}}{S} = \frac{R_H.Tan\Phi_{SL}}{R_v} = \frac{Tan\Phi_{SL}}{Tan(\delta_R - \alpha)} \geq 1.5$$

$$\sum H = E_{ah} + T_H = R_H \qquad \sum V = W + A + E_{av} + T_v = R_v \qquad \delta_R = \arctan\frac{R_v}{R_H}$$

If the foundation base is inclined at an angle α with the horizontal plane, then we have:

$$R'_V = R_V.\cos\alpha + R_H.\sin\alpha \qquad R'_H = R_H.\cos\alpha - R_V.\sin\alpha$$

In case the foundation is placed on rock and the anchor rods are designed adjoining the foundation and the rock, we have:

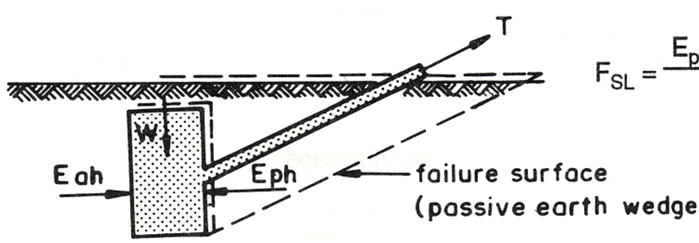
Safety factor with anchor rods, $F_{SL} = \dfrac{R_H.Tan\Phi_{SL} + A_S.\tau_{t.comb}}{R_V} \geq 1.5$

Here, N or R_V the total forces acting perpendicular to the base of the foundation;
R_H the total forces acting parallel to the foundation base;
Φ_{SL} the angle of friction between the subsoil (or rock) and foundation base;
Φ the angle of internal friction of the subsoil.
$\Phi_{SL} = \Phi$ they are generally assumed equal in foundation on soil
$\tau_{.comb}$ the permissible shear stress of anchor rods, (= 0.075 kN/mm^2).
A_S the sectional area of the anchor rods

Deadman Foundation

The process leading to failure in deadman anchorage foundations of LSTB standard bridges is similar to the nature of sliding failure. The deadman foundation mobilizes the weight of the earth mass in front of the foundation, thus producing a resistive force of passive earth pressure to the foundation. But in order to attain peak resistance, a relatively large deformation (1% of the depth of the foundation base) is required. Hence, a high safety factor of $F_{SL} \geq 3.5$ is necessary to reduce this large deformation.

The terminology "deadman" in SSTB bridge foundations is completely different from the above mentioned deadman foundations of LSTB bridges. In the SSTB, the deadman is an anchorage reinforced beam inside the foundation.



$$F_{SL} = \frac{E_{ph} - E_{ah} + (W - T_V)\cdot\tan\Phi}{T_h} \geq 3.5$$

In the LSTB bridge standard, the safety factor against sliding is calculated thus:

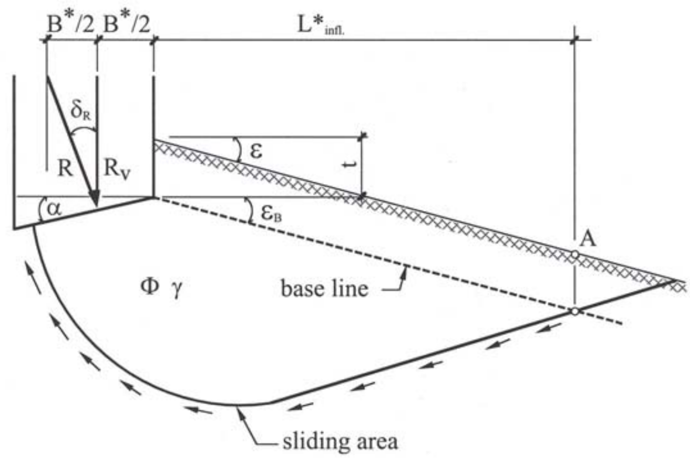
Here, E_{ph} and E_{ah} the passive and active earth pressure acting on the foundation;
T_v and T_H the vertical and horizontal component of the cable tension;
W the weight of the foundation and the load over it and
Φ the angle of internal friction of the subsoil

4.9.3 Safety Against Ultimate Bearing Capacity (Ground Shear Failure)

In our case, the horizontal force acting on the foundation is relatively greater than the vertical load. The safety against ground shear failure or bearing capacity depends mainly on the vertical load on the foundation. Therefore, in a majority of the cases, this failure mode is not the deciding factor in the foundation design. However, there is comparatively high vertical force acting on the walkway and the tower foundation, and ground shear failure may be the deciding factor in its design.

The ground shear failure analysis is carried out by Terzaghi's extended and amplified bearing capacity formula. Terzaghi's formula is extended and amplified with different co-efficients considering the foundation shape and topography of the ground.

The analysis model is as shown in the figure below:



Safety against ground shear failure, $F_{BC} = P^*/P \geq 2.0$ for Terzaghi's formula,

where, P^* - shear resistance of the ground (ultimate bearing capacity) determined by Terzaghi's formula

P - vertical resultant force on the foundation

Terzaghi's formula for shallow foundation with continuous footing and horizontal ground,

$$P^* = B\,[c\,N_C + (\Upsilon\,t + q)\,N_q + 0.5\,B\,\Upsilon\,N_\Upsilon],$$

cohesion factor "**c**" is not considered since the predominant soil type in Nepal is non-cohesive,

therefore, Terzaghi's formula above will be,

$$P^* = B\,[\,(\Upsilon\,t + q)\,N_q + 0.5\,B\,\Upsilon\,N_\Upsilon]$$

In our case, safety against ground shear failure, $F_{BC} = P^*/R_v \geq 2.0$

where, P^* - shear resistance of the ground (ultimate bearing capacity) determined by Terzaghi's formula with correction factors for topography and shape of the foundation,

R_v - vertical resultant force on the foundation

Terzaghi's formula with correction factors,

$$P^* = B^*\,L^*\,[(\Upsilon\,t + q)\,N_q\,S_q\,d_q\,i_q\,b'_q\,g_q + 0.5\,\Upsilon\,B^*\,N_\Upsilon\,S_\gamma\,d_\gamma\,i_\gamma\,b'_\gamma\,g_\gamma]$$

where,

Υ = unit weight of soil

t = embedded depth in front of the foundation

q = surcharge load, $q = 0$ in our case

N_q = $e^{\Pi\tan\phi}\ \tan^2(45° + 0.5\,\phi)$, co-efficient for embedded depth or surcharge load,

$N_\Upsilon \approx$ $1.8\,(N_q - 1)\tan\phi$, co-efficient for effect of the foundation width, shape correction factors considering limited length (L*) of continuous footing,

$$S_q = 1 + (B^*/L^*) \tan \phi$$

$$S_\gamma = 1 - 0.4\,(B^*/L^*)$$

depth correction factors considering the embedded depth ($t = h_p$) of the foundation,

$$d_q = 1 + 0.035 \tan \phi (1 - \operatorname{Sin} \phi)^2 \ \arctan(t/B^*),$$

$$d_\gamma = 1$$

embedded depth in front of the foundation should be guaranteed up to the length of the influence area L^*_{infl},

$$L^*_{infl} = B^* \tan(45° + 0.5\phi)\ e^{0.5\,\Pi \tan\phi}$$

correction factor for the inclination of the load,

$$i_q = [1 - 0.5 \tan(\delta_R - \alpha)\,]^5$$

$$i_\gamma = [1 - (0.7 - 0.0022\,\alpha°) \tan(\delta_R - \alpha)]^5$$

correction factor for the inclination of the foundation base, α,

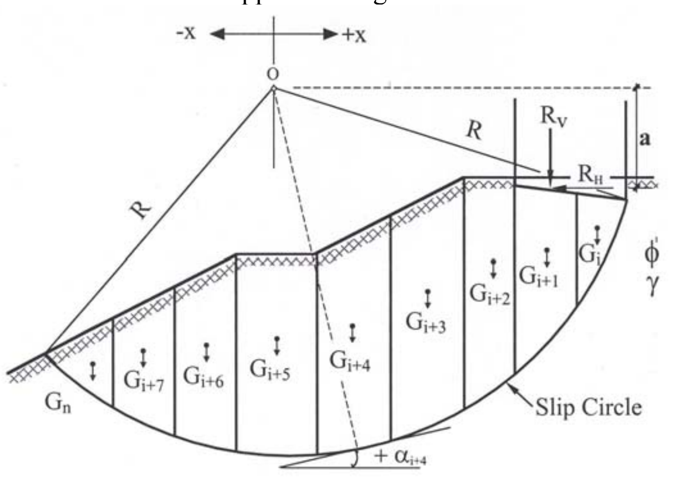
$$b'_q = e^{(-\,0.035\,\alpha° \tan \phi)}$$

$$b'_\gamma = e^{(-\,0.047\,\alpha° \tan \phi)}$$

correction factor for the inclination of the base line (ε_B), which is approximately equal to the inclination of the ground slope (ε) but limited to the ground slope up to 15° only,

$$g_q = g_\gamma = (1 - 0.5 \tan \varepsilon_B)^5, \quad \varepsilon_B \approx \varepsilon$$

Experience shows that Terzaghi's amplified and extended formula with correction factors is no more reliable for foundations on steep slope ground. It gives relatively acceptable results only for a ground with a slope of less than 15°. Therefore, an overall slope stability analysis is made for slope ground of more than 15° inclination. The overall slope stability analysis is carried out by using Bishop's simplified method of slices. The method applied is as given below:



$$F = \frac{\sum_{i=1}^{n} \left[(G_i + V_i) \tan\phi' \right] \frac{1}{m_{\alpha i}}}{\sum_{i=1}^{n} \left[(G_i + V_i) \sin\alpha_i \right] + R_H \frac{a}{R}}$$

$$m_{\alpha i} = \cos\alpha_i \left[1 + \frac{\tan\phi' \tan\alpha_i}{F} \right]$$

where, in our case

G_i = weight of the soil of the given slice "i"

V_i = R_V, resultant vertical force on the foundation, which is applicable only at one slice and in other slices, $V_i = 0$.

H = R_H, resultant horizontal force on the foundation

ϕ' = 1.10 x ϕ,

ϕ = angle of internal friction of the soil

α_i = tangential angle at the slip circle of given slice "i"

R = Radius of the slip circle

a = lever arm of "R_H"

F = safety factor, which is taken as 1.5 for the Bishop's Method

As the factor of safety "F" appears on both the sides, a process of successive approximation is required. It is necessary to fix the center "**o**" corresponding to the critical slip circle with minimum safety factor. A computer program is also available to do this analysis.

As per the analysis, it is established that if the foundations are placed behind the slope line with the angle of internal friction "ϕ", shear failure in general, will not occur.

In our calculation, based on the fact that the horizontal force acting on the foundation is relatively higher than the vertical force and consequently the chance of the foundation sliding is more serious that its settlement and ground shear failure, we limit checking the bearing capacity of the soil to the following:

4.9.4 Safety Against Ground Bearing Pressure

$\sigma_{max} \leq \sigma_{perm}$ (permissible bearing capacity of the soil at site, defined by the soil investigation)

The pressure developed by the rigid foundation can be calculated as:

General Case

1) When the resulting force in the foundation, ***R*** is located within the core of the foundation, the whole base will be under pressure: $\sigma_{1-4} > 0$

$$\left[\frac{e_B}{B} + \frac{e_L}{L} \right] < \frac{1}{6}$$

The pressures at the four corners can be calculated by using the following formulas:

$$\sigma_{1-4} = \frac{R_V}{B.L} \cdot \left[1 \pm 6\frac{e_B}{B} \pm 6\frac{e_L}{L} \right] \qquad \text{kN/m}^2$$

$$\sigma_{\max} = \frac{R_V}{B.L}.\left[1 + 6\frac{e_B}{B} + \frac{e_L}{L}\right] \qquad \text{kN/m}^2$$

$$e_B = \frac{B}{2} - \frac{B^*}{2} = \frac{1}{2}(B - B^*) \qquad \text{m}$$

$$e_L = \frac{L}{2} - \frac{L^*}{2} = \frac{1}{2}(L - L^*) \qquad \text{m}$$

2) When the resulting force in the foundation, R is located outside the core of the foundation

If the resultant loading force is located outside the core of the foundation $\left[\frac{e_B}{B} + \frac{e_L}{L} > \frac{1}{6}\right]$ negative pressure will occur which cannot be transmitted to the soil.

The calculation of the pressure will be quite difficult unless R lies on one of the axes. (Refer to special case).

The maximum ground-bearing pressure must be calculated by introducing the factor, z

$$\sigma_{\max} = z.\frac{R_V}{B^*.L^*} \qquad \text{kN/m}^2$$

The Value of Z

z-factor	(B*/2)/B					
(L*/2)/L	0.50	0.45	0.40	0.35	0.30	0.25
0.50	***1.00***	***1.17***	***1.28***	1.33	1.33	1.33
0.45	***1.17***	***1.30***	1.36	1.39	1.39	1.39
0.40	***1.28***	1.36	1.41	1.43	1.43	1.43
0.35	1.33	1.39	1.43	1.46	1.47	1.47
0.30	1.33	1.39	1.43	1.47	1.49	1.50

The figures in bold and italics are the values when the location of the resultant force lies within the core.

3) Special Case (no lateral forces)

The resultant force is located on the axis parallel to B (e.g., bridge axis)

$\boldsymbol{R_{HL}} = 0$ and $\frac{L^*}{2} = \frac{L}{2}$

and by omitting indices B: $R_{HB} = R_H$ and $e_B = \frac{B}{2} - \frac{B^*}{2} = \frac{B - B^*}{2}$

1) $n < e_B < \frac{B}{6} or \frac{B}{2} > \frac{B^*}{2} > \frac{B}{3}$

$$\sigma_{\max\min} = \frac{R_V}{B.L}.\left[1 \pm 6\frac{e_B}{B}\right] = \frac{R_V}{B.L}\left[1 \pm \left(3 - 3\frac{B*}{B}\right)\right] \qquad \text{kN/m}^2$$

2) $e > \frac{B}{6} or \frac{B*}{2} > \frac{B}{3}$

a) The negative pressure will be borne by another tension member (e.g., reinforcement or anchorage rod).

$$\sigma_{\max\min} = \frac{R_V}{B.L}.\left[1 \pm 6\frac{e_B}{B}\right] = \frac{R_V}{B.L}\left[1 \pm \left(3 - 3\frac{B*}{B}\right)\right] \qquad \text{kN/m}^2$$

b) No negative pressure can be transmitted (e.g., to the soil or dry stone structures), or the impact of the tension-bearing member is neglected.

$$\sigma_{\max} = \frac{2R_V}{3\left(\frac{B}{2} - e_B\right).L} = \frac{2.R_V}{3.\frac{B*}{2}.L} = \frac{4.R_V}{3.B*.L} \qquad \text{kN/m}^2$$

4.9.5 Safety Against Overturning

A monolithic foundation, such as a retaining wall, can be seen in usual practice being toppled around its front base point. The cause is the misbalance of the retaining moment against the driving moment around that point. A gravity foundation at rest must exercise gravitational forces at any point of its base. For this condition, the sum of the retaining and driving moments should be zero only at a certain area around the middle of the foundation base. Hence, a resulting force of all forces acting on the foundation shall lie only at that area. The resultant force can be determined with its line of action and the location of the point on the base of the foundation by equating the sum of the moments as thus:

Taking the moment of the resultant force at the front base point of the foundation, we get:

$$M_F = R_V.\frac{B*}{2}.\frac{1}{\cos\alpha} = R.\cos(\delta_R - \alpha).\frac{B*}{2.\cos\alpha} = \frac{R_V.\cos(\delta_R - \alpha)}{\cos\delta_R}.\frac{B*}{2}.\frac{1}{\cos\alpha}$$

On the ether hand, the moment at the same point exerted by the component forces is:

$$M_F = W.w + E_{av} + A.a + E_{ah}.\left(B.Tan\alpha - \frac{h_a}{3}\right) - T_H.h_T$$

Equating, we get eccentricity:

$$\frac{B*}{2} = \frac{M_F.\cos\delta_R.\cos\alpha}{R_V.\cos(\delta_R - \alpha)} = \frac{M_F}{R_V.(1 + Tan\alpha.Tan\delta_R)}$$

This is the distance from the front of the foundation to the point of action of the resultant force on the bridge axis parallel to the breadth, B direction of the foundation.

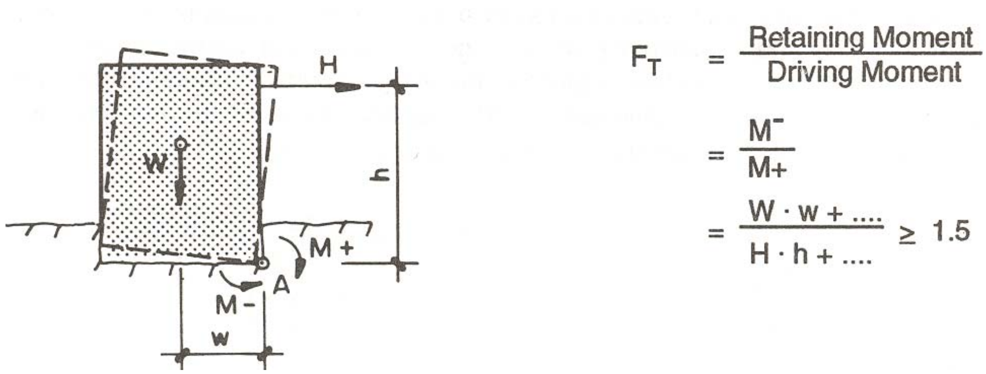
In a similar way, we can get the distance from the side of the foundation to the point of action of the resultant force on the axis perpendicular to the bridge axis, parallel to length, L direction of the foundation.

$\frac{L^*}{2} = \frac{M_F^L}{R_V}$ (Usually, the foundation base on the axis perpendicular to the bridge axis is not inclined and therefore the formula is simplified)

$$M_F^L = W.\frac{L}{2} + \sum F_L . f_L$$

Here, F_L the forces acting on the foundation along the L (length) direction;
f_L the perpendicular distance to the line of forces FL forms the base line at mid-foundation

A foundation with its resultant outside the foundation base or close to its border will start to topple. Comparing the driving and retaining moment at the border of the foundation generally controls this mechanism. Usually, a safety factor of $F_T \geq 1.5$ is required.



$$F_T = \frac{\text{Re}\,tainingMoment}{DrivingMoment} = \frac{M}{M^+} \geq 1.5 -$$

Taking the moment around point F, the bottom point of the front face of the foundation, we get:

$$M^+ = W.w + E_{av}.B + A.a + E_{ah}.\left(Tan\alpha - \frac{ha}{e}\right) \qquad \text{and} \qquad M^- = T_H.h_T$$

$M_F = M^+ - M^- = 0$

Another approach to controlling this failure mechanism is to set limits to the eccentricity of the resultant force in relation to the center point of the foundation base area. As long as the resultant force lies within the core of the foundation base, the whole contact area foundation-soil is subjected to compression so that no gap will develop. This restrictive requirement is applied for important foundations. In most cases, a gap of about one-third to one-half of the base area is tolerated.

4.9.6 Rock Anchors

1. General

Anchorage rods can be provided to prevent the foundation from sliding and/or toppling. The anchorage rods should be placed at the back of the foundation to make sure that they are embedded in sound rock.

Specifications for anchorage rods

High tensile steel (ribbed reinforcement steel bars) (refer to 4.3.4).

Permissible tension stress	$\sigma_{tper} = 230$	N/mm^2
Permissible bond stress	$\sigma_{Bper} = 0.6$	N/mm^2
Permissible shear stress	$\tau_{perm} = \dfrac{\sigma_{tperm}}{\sqrt{3}} = 130$	N/mm^2
Combined stress (tension and shear)	$\sigma_{comb} = \sqrt{\left(\sigma_1^2 + 3\tau^2\right)}$	

Diameter of anchorage rods:	d	=	25 mm
Length to be anchored:	la	=	2000 mm
Diameter of hole:	D	=	34 mm
Distance between two anchorage rods:	e_{min}	=	1500 mm

During sliding, the shear resistance, and during over-toppling, the tensile resistance of the anchorage rods will be mobilized.
It will not be possible to mobilize the full shear resistance of the rods during sliding. Therefore, because of practical reasons, it is recommended that the permissible shear stress be reduced to:

$$\tau_{tcombined} = 75N / mm^2$$

In order to fulfill the formula for combined tensile and shear stress:

$\left(\dfrac{\sigma_{tcomb}}{\sigma_{tperm}}\right)^2 + \left(\dfrac{\tau_{comb}}{\tau_{perm}}\right)^2 \leq 1$ the permissible tensile stress has to be reduced to:

$$\sigma_{tcomb} = \sigma_{tperm} \cdot \sqrt{1 - \left(\frac{\tau_{comb}}{\tau_{perm}}\right)^2} = 230 \cdot \sqrt{1 - \left(\frac{75.\sqrt{3}}{230}\right)2} = 190N / mm^2$$

The necessary anchorage length needed in order to develop the bond resisting the tension may be calculated as follows:

$$T_{t\max} \leq A_S .\sigma_{tcomb} = \frac{d^2\pi}{4} .\omega_{tcomb}$$

$$l_a \geq \frac{T}{n.d.\sigma_{Bperm}} = \frac{\frac{d^2\pi}{4}.\sigma_{tcomb}}{n.d.\pi.\sigma_{Bperm}} = \frac{d.\sigma_{tcomb}}{4.n.\sigma_{Bperm}}$$

Therefore: $1_{\alpha\min} \geq \dfrac{25.190}{4.1.0.6} = 1980mm \cong 2000mm(2meter)$

2. Calculation of the Number of Anchorage Rods

No anchorage rods are necessary if $B^*/_2 > B/_2$, as the whole foundation base is under pressure. A minimum number of anchorage rods is necessary if $B^*/_2 < B/_3$

As a first step, the bearing stress distribution is calculated on the uncracked cross-section and the theoretically required cross-section of the anchorage rods is determined under the assumption that the tensile stresses are taken over by the anchorage rods:

$e > B^{*}/_{6}$ or $B^{*}/_{2} < B/_{3}$

$$\sigma_{\max\min} = \frac{R_V \cdot}{B \cdot .L} \pm \frac{6R_v \cdot .e \cdot}{B \cdot^{2} .L} \qquad \text{kN/m}^2$$

$$x \cdot = \frac{B \cdot .\sigma_{\min}}{\sigma_{\min} - \sigma_{\max}} \qquad \text{m}$$

$$A_S = \frac{1\sigma_{\min} 1.x \cdot .L}{2\sigma_{tcomb}} \cdot \frac{a}{b} \qquad \text{mm}^2$$

$$a = B \cdot -B^{*}\ 1\!\!\diagup\!\!_2 - x\ 1\!\!\diagup\!\!_3 \diagup \qquad \text{m}$$

$$b = B - s - B^{*}/_{2} \qquad \text{m}$$

$$N = k\frac{4As}{d^2 \pi}$$

Correcting Coefficient, k

Rock types and fractures	k-Value
Plutonic rock, gneiss, quartzite, hard sandstone, massive dolomite and limestone, not weathered, few fractures	
Quartzite, gneiss, massive limestone and dolomite, $\pm$ fractured	1.75
Phyllite, crystalline schists, not weathered	2.00
Weathered schists and phyllite, thin-bedded limestone and dolomite, calcshists, slates	2.25

(The value takes into account the type of and fractures in the underlying rock)

Rock formations are usually more or less structured by a number of weakness planes caused by cracks and fissures. Cracks and fissures may be closed or open and appear close or at a distance. A rock formation loaded with additional loads, e.g., a bridge foundation, may be subject to motion along these weakness planes. Additional rock anchorages must be provided to avoid this.

The safety factors and the requirements for foundations on soil or strongly weathered rock are as follows:

Safety Factor	Walkway / Tower Foundation	Main Cable Foundation	Main Anchorage Foundation	Windguy Cable Foundation	Retaining Wall & Gabions
F_{SL}	≥ 1.5	≥ 1.5	≥ 1.5	≥ 1.5	≥ 1.5
F_{BC}	≥ 2.0	≥ 2.0	≥ 2.0	≥ 2.0	≥ 2.0
σ_{max}	$< \sigma_{perm}$	$< \sigma_{perm}$	$< \sigma_{perm}$	$< \sigma_{perm}$	$< \sigma_{perm}$
F_T	≥ 1.5	≥ 1.5	≥ 1.5	≥ 1.5	≥ 1.5
$B^*/2, L^*/2$	$\geq \frac{B+E}{3}, \geq \frac{L}{3}$	$\geq \frac{B}{4}$	$\geq \frac{B}{4}$	$\geq \frac{B}{4}$	$\geq \frac{B}{4}$
F_{slope}	Depending on method ($\geq$ 1.3 to 1.5)				
α (max)	0^o	$\leq 15^o$	$\leq 15^o$	$\leq 15^o$	$\leq 15^o$
Anchor rods for stabilizing foundation	none	none	No. as per calculation	No. as per calculation	None

The safety factors and the requirements for foundations on rock are as follows:

Safety Factor	Walkway/ Tower Foundation	Main Cable Foundation	Main Anchorage Foundation	Windguy Cable Foundation	Retaining Wall & Gabions
$F_{SL\ without\ rods}$ $F_{SL\ with\ rods}$ $F_{SL\ neglect\ rods}$	≥ 1.5 / /	≥ 1.5 / /	≥ 1.5 ≥ 1.5 ≥ 1.3	≥ 1.5 > 1.5 ≥ 1.5	≥ 1.5 / /
	$\leq \sigma_{perm}$ (from survey)				
$F_{T\ without\ rods}$ $F_{T\ with\ rods}$ $F_{T\ neglect\ rods}$	≥ 1.5 / /	≥ 1.5 / /	≥ 1.5 ≥ 1.5 ≥ 1.2	≥ 1.5 ≥ 1.5 ≥ 1.2	≥ 1.5 / /
$B^*/_2, L^*/_2$	$\geq \frac{B+E}{3}, \geq \frac{L}{3}$	$\geq \frac{B}{6}$	$\geq \frac{B}{6}$ $\geq \frac{B}{3}$ without rods	$\geq \frac{B}{6}$	$\geq \frac{B}{3}$
Slope	Stability of rocky slopes to be checked, refer to Volume B 5.41				
α (max)	0^o	$\leq 18^o$	$\leq 18^o$	$\leq 18^o$	$\leq 15^o$
Anchorage Rods for Stabilizing the Foundation	None	None	Numbers according to calculation, or minimum	Numbers according to calculation, or minimum	none