THE DESIGN AND CONSTRUCTION OF LIGHT SUSPENSION BRIDGES.

By A. St. G. LYSTER,

Executive Engineer.

Introductory—Literature—Nomenclature—Traffic—Types—Unstiffened Bridges—Main Cables—Towers—Saddles—Anchorages—Suspenders—Transoms—Roadway—Camber—Wind ties—Cradling—Inverted Catenary—Alternate Cable Suspension—Stiffened Roadway—Stays—Stiffening Truss—Stiffened Cables.

Introductory.

The suspension bridge in India has been developed almost entirely in the hills, where the depth and velocity of the water in the rivers renders the erection of centring impossible. The design has naturally been affected by the light nature of hill traffic and the difficulty of importing materials in a mountainous country, and it is with the suspension bridge as evolved in the Himalayas that this paper is concerned. "Light" suspension bridges as here understood are not intended to carry wheeled traffic.

In this paper the author does not propose to deal with the theory of suspension bridges, which can be found in text books, but to discuss some of the details of design in their relation to construction.

The author is very much indebted to Mr. A. R. Astbury, who generously placed at his disposal a large collection of notes on the subject.

Literature.

The literature of the subject may be briefly tabulated as follows :—

1. P. W. D. Paper 51. Type calculations with numerical example of a span of 150 feet.
2. P. W. D. Paper 54. Hints and tips on the erection of suspension bridges.
3. P. W. D. Paper 59. The Dehar suspension bridge.
4. Notes and Calculations for the Vishnuprayag suspension bridge.
5. A number of papers in the R. E. Journal dealing chiefly with expedients and bridges designed for rapid erection in the field.
6. Military Engineering, Part III A. and III B.

A list of Himalayan suspension bridges is given in Appendix A.

Nomenclature.

The following nomenclature will be observed throughout the paper. Symbols in small or italic type are used with different meanings which are explained as they occur :—

S=Span. Horizontal distance between points of support of cable.

D=Dip. Vertical distance between points of support of cable and lowest point of cable.

$\frac{D}{S}$=Dip ratio.

L=Length of roadway between towers.

B=Width of roadway at base of hand-rails.

T_1, T_2, etc.=Tension in main cables at any indicated point.

Backstay=Portion of main cables between towers and anchorage.

Suspenders=Connections between roadway and main cables.

Stays=Inclined rods or ropes from tops of towers to different points on roadway.

Wind ties=Any fastening between the roadway and the bank of the river below road level.

Cables "cradled"=Point of attachment of suspenders to transoms so adjusted as to make the cables and suspenders lie in a plane inclined at an angle to the vertical.

Bay=That part of the structure which lies between one transom and the next.

Traffic.

A "light" suspension bridge has already been defined as one which is not intended to carry wheeled traffic; it is then only necessary to provide for pack animals and foot passengers.

Military Engineering, Part III A, gives the following tables of weights :—

	Weight in lbs.					Dimensions.	Area occupied on bridge.	Weight per square foot.
	Animal.	Load.	Total.	On hindlegs.	On forelegs.			
							Sq. ft.	lbs.
Pack horses ..	1,400	200	1,600	640	*960	..	..	..
Pack mules ..	1,000	160	1,160	460	700	..	..	..
Pack bullocks ..	450	150	600	200	400	5′ × 2′ 9″	13·5	44
Pack elephants ..	7,000	1,500	8,500	3,500	5,000	11′ × 9′	100	85
Pack camels ..	1,200	400	1,600	600	1,000	10′ × 7′	70	23
Commissariat cattle ..	1,000	..	..	..	..	..	10	100

*This may be taken as the greatest weight that can be brought on to any one foot.

The weight of infantry in marching order assuming they are crowded at a check to the greatest extent possible short of losing their formation can be taken as follows :—

	Weight per foot run of bridge.	Required width of road.	Weight per square foot of road
	lbs.		lbs.
Single file ..	140	3 feet wide	46·6
File ..	280	6 feet wide	46·6
Fours ..	560	8 feet wide	70·0

Infantry in marching order .. Average 200 lbs. per man . . A

Infantry in unarmed order .. Average 160 lbs. per man.

When infantry lose their formation and crowd together in a disorganised mass, they may bring the following weight on the bridge :—

Marching order .. 133 lbs. per square foot.

When unarmed .. 175 lbs. per square foot .. B.

A laden coolie may be taken as equivalent to A.

Maximum possible crowd load may be taken as equal to B. It is now evident that the live load which we have to provide for depends directly on the width of the roadway. This width depends entirely on the density of traffic to be carried. A not uncommon device in the Sutlej valley is to design the roadway so that a single file of laden mules blocks it for all other traffic.

SECTION

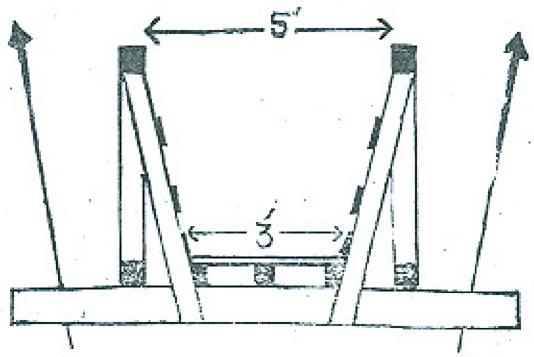


Fig. 1.

This design very definitely limits the live load, and is suitable to localities where conflicting lines of traffic are not likely to meet at the bridge or where delay is of small importance. It is difficult to lay down the width occupied by a mule and its loads ; certainly a 6-feet road (a very common width on the Sutlej Bridges) does not allow two laden mules to pass each other, but makes it possible for man if not carrying a very bulky load to pass a laden mule, and allows of dense crowding by laden animals if checked on the bridge.

For bridges with a 3-feet roadway as described above 50 lbs. per square foot of roadway is probably a reasonable allowance for live load, and for a road width of 6 feet and more a live load of 100 lbs. per square foot may be experienced. On inspection of the calculations made for existing bridges it is found that the equivalent dead load allowed for in place of live load averages from 60 lbs. to 100 lbs. So that in these bridges when maximum loading occurs impact stresses are only provided for by the factor of safety. That is to say that the bridges have actually a lower factor of safety than the designer intended. Having regard to the infrequency with which maximum loading is likely to occur, it is not unreasonable to depend somewhat on the factor of safety in providing for it.

A fair allowance for an average bridge in the upper part of the Sutlej valley would be :—

Dead load=dead weight of structure=50 lbs. per square foot of roadway.

Live load=(equivalent dead load)=100 lbs. per square foot of roadway.

150 lbs. per square foot of roadway.

On the other hand in designing a bridge situated near a town it would be unwise to curtail the allowance for load in any way.

Types.

The next step after providing for the traffic is to select the type of bridge to be built.

Suspension Bridges may be divided into two main classes :—

I.—Unstiffened.

II.—Stiffened.

In type I a moving load is transferred direct to the cables by each suspender in turn.

In type II a moving load is transferred to the cables through the medium of trusses.

Type II divides readily into classes (*a*) Stiffened roadways ; (*b*) Stiffened cables.

It will be convenient while discussing Type I to examine many details of construction which apply equally to both types.

Unstiffened Bridges.

An unstiffened suspension bridge includes the following members :—

The main cables ; the towers ; the anchorages ; the suspenders fastened to the main cables by strong clips and which support the transoms ; on the transoms, and connecting them one to another, are road beams, and on these are laid the floor of the bridge ; a hand-rail consisting of uprights and longitudinals (no cross bracing) is given on each side of the road, and some form of wind tie completes the structure.

Note that the road beams are jointed at each transom so that the roadway is a more or less flexible chain.

Let us examine these members in turn.

Main cables.

A rope hanging freely under its own weights falls into the curve known as a catenary, but when loaded with a uniformly distributed horizontal load of a magnitude much greater than the weight of the rope the curve becomes a parabola. This condition may be said to exist for all light suspension bridges.

Take the case of a wire hanging freely under its own weight

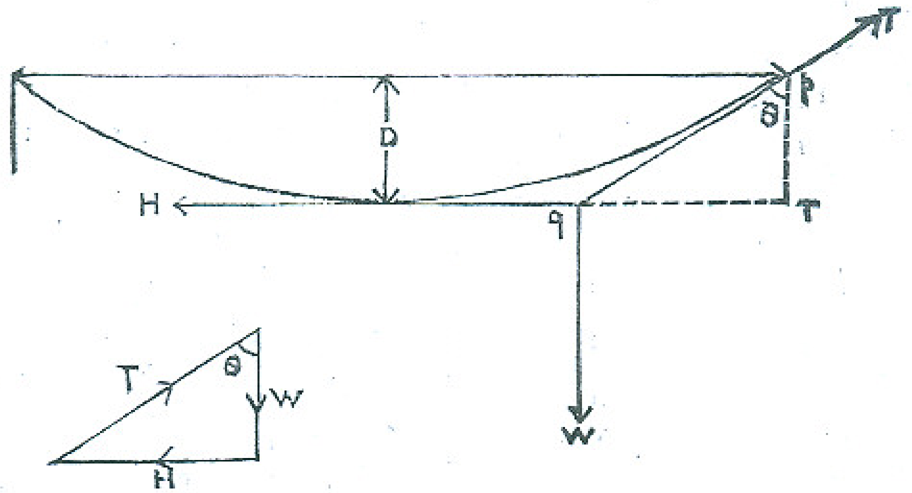


Fig. 2.

W=weight of half span of wire.

w=weight of wire per unit length.

H=the horizontal component of T (the tension at any point) is constant throughout the span.

Considering the half span we have three forces H, W, and T, in equilibrium.

$$\therefore T = W \text{ Sec } \theta \quad \text{Sec } \theta = \frac{pq}{pr} = \frac{pq}{D}$$

$$W = w\sqrt{\left(\frac{S}{2}\right)^2 + \frac{4}{3}D^2} \text{ approx. formula.}$$

If S remains constant and D is varied.

D may be varied from O to infinity.

W will vary from $w\frac{S}{2}$ to infinity.

Sec θ varies from infinity to 1.

D	W	Sec θ
O	$w\,\frac{S}{2}$	Inf.
Inf.	Inf.	1.

It is clear that T is infinitely great for the two limiting values of D, but that T has an intermediate minimum value.

The minimum value of T is found to occur when D=0·34 S

or when $\frac{D}{S} = \frac{1}{3}$

This, however, is neither the most suitable nor the most economical dip ratio to use for bridge work. The use of $\frac{D}{S} = \frac{1}{3}$ would entail very lofty and expensive towers, and the greater the value of D, the greater the tendency to swing under wind pressure, and the greater the deformation under concentrated loads.

In practise a ratio $\frac{D}{S}$ is usually chosen between $\frac{1}{10}$ and $\frac{1}{15}$, the smaller value being more suitable for unstiffened bridges.

In a bridge the horizontal load far exceeds the weight of the cable, so that in the equation T=W Sec θ within practical limits of variation, W is constant, and T varies as Sec θ, for different values of D.

Illustration for total load of 150 lbs. per square foot, span 200 feet.

S.	B.	D.	W.	T.	$\frac{D}{S}$
ft.	ft.	ft.	lbs.	lbs.	
200	6	20	$150 \times 6 \times \frac{200}{4}$	122,000	$\frac{1}{10}$
200	6	13·3	$150 \times 6 \times \frac{200}{4}$	175,500	$\frac{1}{15}$

It will be evident from the above that any change in the length of a cable, such as would be caused by a change in temperature, will by altering D cause a variation in the maximum tension in the cable. Assuming that the adjustment of the cable in erection is done at or near mean temperature (a condition not difficult to fulfil), the maximum change of temperature will be about 50° Fahr.

In a bridge whose total cable length is 410·4 feet with a centre span of 200 feet, this will cause an elongation of cable of 0·123 feet only, and even if the whole of this takes effect in the centre span, the resulting change in dip will be negligible.

The manufacturers of the wire ropes which are commonly used as main cables for light bridges describe their wares under a variety of names, such as "Plough Steel," "Improved Plough Steel", "Special Improved Plough Steel." These terms indicate different strengths of steel ropes of any given diameter and manufacture, but it is necessary to give other details when specifying steel ropes in an order.

Messrs. George Cradock & Co., Calcutta, were asked by the author to specify steel wire rope for main cables of a suspension bridge, each rope to have a breaking strength of $\begin{matrix}50\\75\\100\end{matrix}$ tons.

They replied as follows :—

"The term "Plough Steel" or "Extra Plough Steel" is not a statement of the quality of the steel used in the rope. It is nothing more than a name used for steel of a certain tensile strength. Plough steel wire may be manufactured from the lowest grade used in rope making, namely basic grade.

When this grade of steel is drawn up to a high tensile strength, flaws occur in the wires, and it is therefore not to be recommended for any work where shocks and severe bending stresses occur in the rope.

For ropes manufactured of steel of a tensile strength of over 90 tons per square inch acid grade or special acid is to be recommended. Steel of this quality loses none of its resilience and ductility when drawn up to high tensile strengths.

For plough steel ropes which have a tensile strength of 100—110 tons per square inch of steel, we should recommend special acid grade steel if they are to be used for winding, but where the rope is more or less stationary as in suspension bridge work, acid grade is quite suitable.

As regards the construction to be used, we are of the opinion that a rope of 6/19 combination, owing to the very small section of the wires, does not offer sufficient resistance to corrosion. In practice we have found that the best type of rope to employ is locked coil or spiral construction. These, however, are not very flexible, and in your case (light suspension bridge) we would suggest a rope of 6/7 combination with a central wire core making in all 7 strands of 7 wires each. This is sufficiently flexible to allow of ease in handling, and further is cheaper than 6/19 combination, and also gives much greater resistance to external and internal corrosion. We recommend a wire core because if a hemp core is used there is a tendency for the rope to flatten at the saddles, thereby opening up the strands and allowing wet to permeate the core and rot this. A rope with a hemp core is also easily crushed out of shape by the application of clamps and suspension rod attachments."

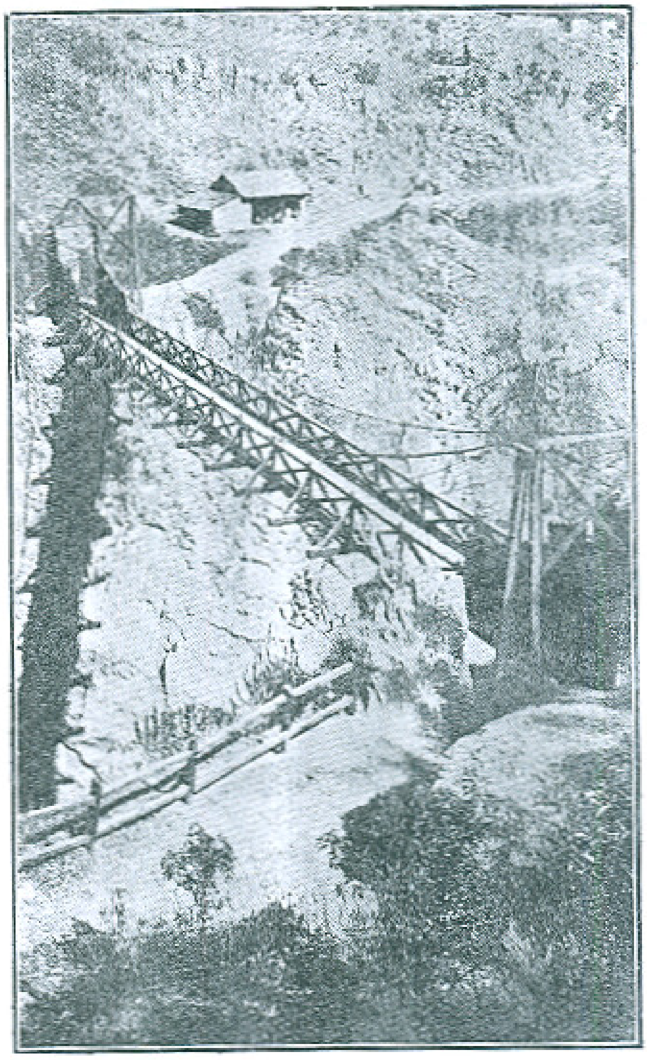
They recommend the following ropes :—

For 50 tons ..	3¾″ Circ. Special Improved Patent Steel.	Strength per sq. inch 90/100 tons.	Acid grade.
For 75 tons ..	4½″ Circ. Special Improved Patent Steel.	Strength per sq. inch 90/100 tons.	Acid grade.
For 100 tons ..	5″ Circ. Best Plough Steel.	Strength per sq. inch 100/110 tons.	Special acid grade.

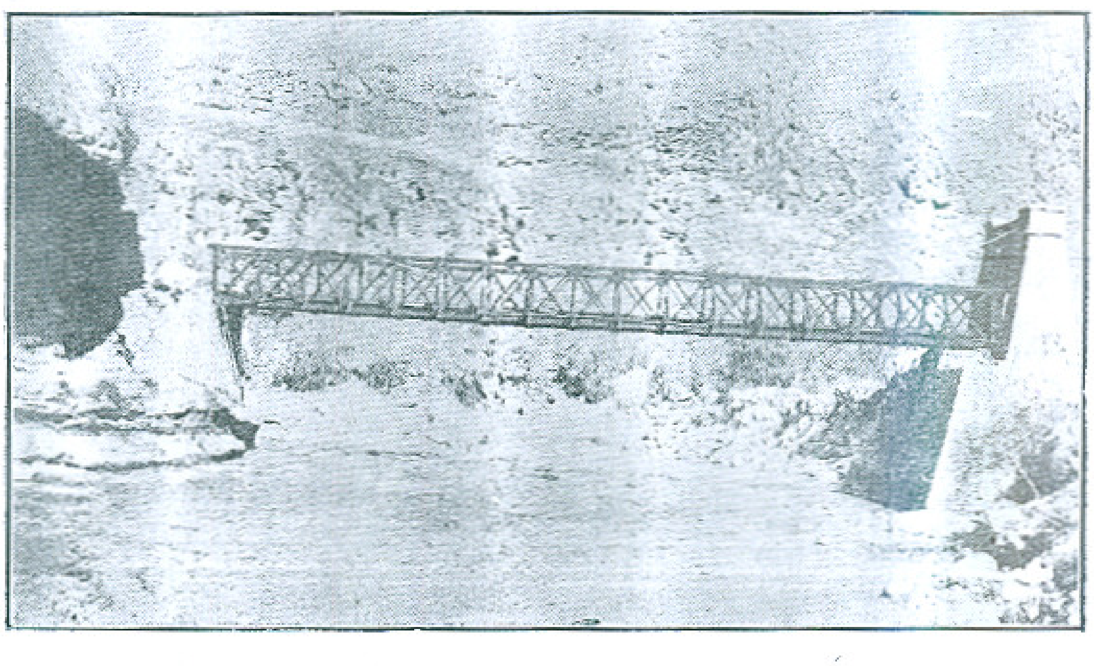
Towers.

The design of the towers does not call for much comment. They are usually built of masonry because the materials are obtainable locally. The backstays are usually arranged so as to cause the resultant pressure on the towers to be vertical, but if this is not feasible, it is a simple matter to design towers to withstand a resultant pressure which is at an inclination to the vertical. Steel towers are to be avoided in the Himalayas. The Wangtu bridge has steel lattice towers, and wandering Thibetan traders have succeeded in removing the rivets and taking away several consecutive pieces of the lattice bracing. This has weakened the tower which has been slightly bent by the pull of the main cables. This bending would not have occurred if the saddle had been correctly designed. The

SCHEDULE A.



SUNI BRIDGE.



NOGLI BRIDGE.

Suni Bridge has steel towers built up of inclined steel beams, there are no rivets or small pieces for hillmen to pilfer, but the design is crude and inartistic, and is not recommended on those grounds.

Saddles.

On the top of the tower is bolted a bearing plate for the main cables called the saddle. Three designs are shown in Figs. (*i*), (*ii*), (*iii*) on Plate I. The design of saddles as constructed shows that the essential features of this member are not always understood.

It is generally desired to design so that the resultant pressure of the cables on the towers shall be vertical. To obtain this effect two conditions must be fulfilled—

1. The backstay and the tangent to the curve of the span must make equal angles with the horizontal.
2. The tension on both sides of the saddles must be equal.

These two conditions must be severally and independently fulfilled to secure a vertical resultant. Referring to Plate I—

Fig. (*i*) is simply a cast iron block bolted to the tower; there is no provision to eliminate friction or to ensure equal tension through the rope, and the design is only suitable to towers which have been designed for a resultant pressure at an inclination to the vertical. This type was used on Wangtu Bridge, and though the angles of backstay and cable are equal, the variations in tension due to temperature and other changes have been sufficient to cause bending in the weakened tower.

Fig. (*ii*). This type provides for the elimination of friction, but is unsuitable for use when the angles on either side are not equal. This type was used on the Rampur Suspension Bridge, where the angle of the backstay does not equal the tangential angle, with the result that the cast iron block has run forward until brought to rest by the L iron frame which carries the rollers, this frame being thus subjected to a stress it was never designed for.

Fig. (*ii*). This is an improved type equally suitable to all conditions. The rocker will move so as to equalise tensions, and the resultant, which will then be vertical if the angles of the cables backstay are equal, will in any case pass through the axle pin, and so be transmitted to the tower.

The grooves in the saddle should be made to fit the diameter of the cables so as to avoid flattening, the pressure on the saddle being usually in the neighbourhood of 20 tons. The saddle should be carefully set in the line of the cables which is not always parallel to the axis of the bridge.

Anchorages.

The only point calling for special mention in the design of anchorages is the method of attaching the cable.

Two methods are shown, Plate I, Figs. (*iv*), (*v*).

Their respective merits are as follows :—

Fig. (*iv*). It often happens during construction that the exact position of the anchorage has to be altered to suit some local condition of ground or was not accurately known at the time when the design was prepared.

Type (*iv*), with its long overlap of cable, allows considerable latitude for alterations of position, and further adjustments of cable of considerable extent can be made during erection by allowing the cable to slip through one clip at a time. On the other hand cable is an expensive item and unnecessary overlap is to be avoided.

Fig. (*v*). This is a much more economical design, and is suitable to occasions where the precise location of anchorages is known beforehand. It involves the use of lead or solder to which a special danger attaches. In one of the Sutlej valley bridges the fastening was of the type shown in Fig. (*iv*), the loose end of the cable being brought back through a galvanised iron pipe which had already been slipped over the backstay and leaded up. The lead excited the cupidity of some local traders, who proceeded to extract it by lighting a fire underneath, treatment which very nearly wrecked the bridge. If lead is used in the design, it must be made absolutely inaccessible.

Suspenders.

The suspenders or suspension rods are fastened to the main cables by clips. Where there are four cables on each side of the bridge, it is often preferred to clip the cables together in pairs so as to secure equal loading; there will then be two suspenders, one from each clip to the same point on the transom. The suspenders may be either wire rope or steel rods. If the latter are used, they should have a ring joint immediately below the clip. This is especially necessary when the cables are cradled, as otherwise the rods will be subjected to bending stresses when they are only designed for pure tension.

Transoms.

May be either wood or metal. Wood is always subject to decay, and cannot easily be looked after and inspected under the floor. A stirrup fastening for a rolled steel transom is shown on Plate I, Fig. (*vi*).

Roadway.

Usually constructed of wooden road beams and planks. P. W. D. Paper No. 54 deprecates fastening the planks to the road beams, as water lodges between the two causing rot to set in. It is doubtful whether the absence of screws or similar fastenings would make any difference, and loose planks in the roadway of a light suspension bridge which is not a perfectly rigid platform are certainly undesirable. Caution must be exercised in using steel trough flooring or any inherently stiff roadway in an unstiffened bridge. The bending movements set up by live load will be investigated under the heading " Stiffened Roadways.' It is sufficient at this point to enunciate the condition :—

The roadway of an unstiffened suspension bridge must be so designed as to be incapable of transmitting bending stresses from one bay to the next.

The same condition must be observed in the design of the hand rail.

Camber.

It is usual to give camber in the roadway of suspension bridges. Military Engineering, Part III A, page 12, says :—

"In order to allow for settlement in the connections of field bridges and to check progress on to a bridge as well as to facilitate exit from it, it is advisable to make the centre of a bridge higher than the ends, or, in other words, to give camber." Books on theory seldom make any mention of camber, while books and papers on construction nearly all advocate it for no specified reason.

The paragraph quoted above occurs at the end of a section on "Parts common to all bridges," and the writer undoubtedly had in mind the heavy limbered carriages peculiar to military traffic.

The real function of camber is to compensate for deflection of the roadway under live load, so that a concentrated load moving across the bridge actually moves in a horizontal path instead of being always climbing a slope due to the deflection. On this hypothesis the amount of camber required at any point is a calculable quantity, being equal to the corresponding deflection due to maximum load.

Wind ties and other devices for preventing movement in unstiffened bridges.

The great drawback of the unstiffened type of bridge is its flexibility which admits of movements in the roadway.

These movements are of two kinds due to distinct causes—

1. Deformation due to concentrated or partial live loads. This causes a vertical movement of the roadway resulting in undulations.

2. Lateral movement due to wind pressure which causes swaying.

The remedies to be applied to 1 are dealt with under the head "Stiffened bridges."

Cradling.

Against wind pressure, there are a number of devices—(*a*) cradling of cables, (*b*) wind ties, (*c*) horizontal stiffening.

(*a*) *Cradling of cables.*

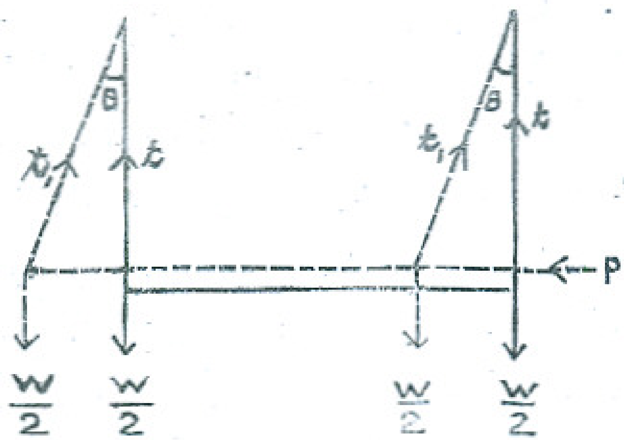


Fig. 3.

Fig. 3 shows the effect of wind pressure when the cables and suspenders hang in a vertical plane.

Before movement takes place, $t=\frac{W}{2}$.

There are no horizontal forces to resist a lateral movement.

Under an applied force P

$t_1=\frac{W}{2}\text{Sec }\theta \quad P=t_1 \sin\theta \therefore P=\frac{W}{2}\tan\theta$

Illustration.

Roadway 6 feet wide. Bay=5 feet, w=150 lbs. per square foot.

Area exposed to wind $2' \times 5=10$ square feet. p=40 lbs. per square foot.

Assuming that P is evenly divided between the two suspenders $P=\frac{400}{2}$; $\frac{W}{2}=2250$ lbs ; $\Theta=10°\cdot21'$; t_1 2275 lbs.

Increased tension T_1 in main cables $=T \text{ Sec } \theta = T \times 1{\cdot}015$.

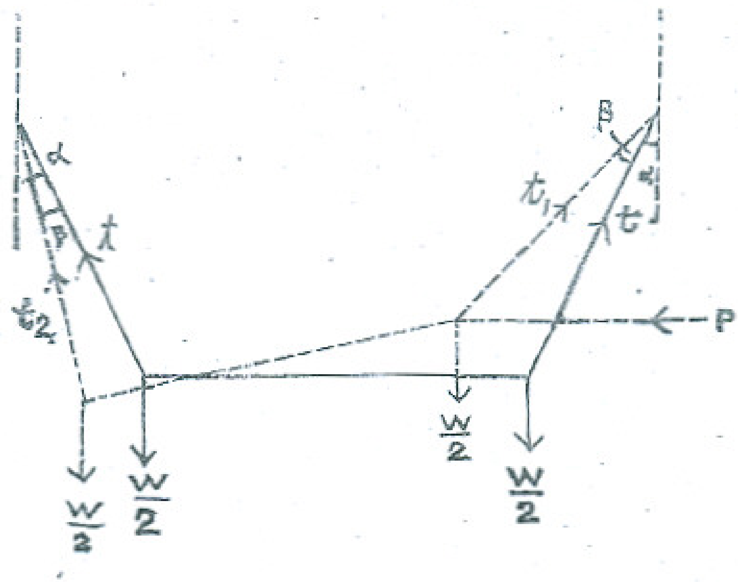


Fig. 4.

Fig. 4 shows the effect of wind pressure when the suspender and main cables are drawn in at an angle α to the vertical.

Before movement takes place

$T=\frac{W}{2}$ Sec α, and there is a horizontal component of $t=h$

$h=\frac{W}{2}$ Tan α, and is balanced by an equal and opposite force at the other end of the transom, this member being thus in a state of tension.

Under an applied force P the suspenders are deflected through an angle β, so that on one side the inclination to the vertical is $(\alpha+\beta)$ and on the other side $(\alpha-\beta)$. The horizontal components h_1 and h_2 of t_1 and t_2 are now unequal, and the difference between them is equal to the applied force P.

$$t_1=\frac{W}{2}\text{Sec}(\alpha+\beta) \quad t_2=\frac{W}{2}\text{Sec}(\alpha-\beta).$$

$$P=h_1-h^2=\frac{W}{2}\left\{\text{Tan}(\alpha+\beta)-\text{Tan}(\alpha-\beta)\right\}$$

Illustration with the same data as before.

P	$\frac{W}{2}$	α	t	t_1	t_2	β	Increased tension in main cables. T Sec $(\alpha + \beta)$.
400	2250	10°	2280	2330	2260	5°	T × 1·035
400	2250	20°	2390	2470	2330	4°·30′	T × 1·099
400	2250	30°	2595	2710	2510	3°·50′	T × 1·204

It is apparent from the above that by cradling the cables to an inclination of 10° to the vertical, the lateral oscillation due to wind pressure can be reduced to about one half at the cost of an increase of tension in the suspenders and main cables of only 3·5%. It does not, however, appear advisable to increase the angle of inclination to 30°, as the results do not justify an increase of 20·4% in the strength of the cables.

In considering the effect of cradling it must be remembered that the wind does not usually act with a steady pressure throughout the bridge, but is in the nature of a partial live load causing deformation of the structure in addition to the lateral movement calculated above.

There are obvious difficulties in construction where the cables consist of rigid eyebars connected by pin joints, as is very frequently the case with large bridges for heavy traffic, but there does not seem to be any very great complication of the design in a light suspension bridge whose cables are of flexible steel rope.

The common practice is first to design the masonry towers on the river bank so that a 6-feet road passes between them. This fixes the saddles at about 12 feet apart. The points of attachment of the suspenders to the transoms are placed as near the road as the design conveniently admits, and the resultant cradling is accepted as suitable. In a design for a bridge of 140 feet span this produced an inclination to the vertical of 14° 50′, which the designer passed over without any attempt to calculate the effects. This is a fortuitous method which has nothing to recommend it. It is better workmanship to select an angle of inclination as suitable to the structure in hand, and from that to find the horizontal distance between the saddles.

Simple wind ties.

We have seen that cradling is only practically effective in preventing lateral movement. A device very commonly adopted is to fasten some point on the roadway (such as the end of one of the transoms) to a fixed point on the bank of the river. This point on the river bank is usually selected to one side of the bridge and below the road level.

It is evident that the tension in such a wind tie can be resolved in two different planes, vertical and horizontal.

In the vertical plane this form of wind tie has a downward component in opposition to the upward pull of the suspenders. It needs no elaborate argument to show that this vertical component can offer no effective resistance to lateral movement ; its effect on the undulatory movement due to live loads will be considered later.

In the horizontal plane there are two components—"Longitudinal," parallel to the axis of the bridge, and "Lateral" at right angles to the axis.

The longitudinal component clearly can offer no resistance to lateral movement : indeed this component, which in many cases noticed by the author has far the largest share of the strength of this form of wind tie, serves no useful purposes whatever ; it introduces a compressive stress into the roadway, which in an unstiffened bridge is not specially suited to withstand it, and since a wind tie to be effective must be taut, *i. e.*, in an initial state of tension, it causes very considerable local stresses at the point of its attachment. Thus the only useful component against wind pressure is the lateral-horizontal component, which in this form of wind tie is usually the smallest.

Continuous Wind Ties.

The compressive stress due to the longitudinal component referred to above can be eliminated by making the wind tie continuous from bank to bank. In light suspension bridges the wind ties are usually constructed of steel wire rope. If such a rope were fastened in a straight line along the ends of the transoms, it would have little effect. We know from the theory of the main cable that the uniformly loaded catenary becomes a parabola. It follows then that the continuous wind tie to offer effective resistance to lateral movement must be designed in the form of a parabola, and since the lateral horizontal component is the only useful one for our purpose, it follows that this parabolic wind tie should lie in a horizontal plane. The principles governing its design are identical with those governing the design of the main cables of the bridge.

Such a wind tie, having no component in the vertical plane, offers no resistance whatever to undulations of the roadway, and is defective against lateral movement in that a partial live load such as is caused by a gust of wind will cause deformation of the parabola because the load is not uniformly distributed along the catenary. This can be provided for by introducing a horizontal stiffening truss designed to distribute partial loading. The design of such a truss is identical with the design of similar trusses to stiffen the roadway against vertical movement, and will be considered in the same section. It is doubtful whether a horizontal truss can be suitably introduced in an unstiffened bridge which is free to make considerable movements in the vertical plane.

This completes the consideration of Type I.—Unstiffened bridges, but before passing to the discussion of Type II.—Stiffened Bridges it will be convenient to consider two devices which do not clearly belong to either class of structure.

Inverted catenary.

In certain suspension bridges in India inverted catenary guy ropes have been used with the object of stiffening the structure.

If this catenary is placed in the vertical plane, it is clear that having no lateral component it can offer no resistance to movement due to wind pressure, and that the initial tension is in direct opposition to the upward pull of the bridge suspenders, thus increasing the tensions in the suspenders and main cables. As used, however, the inverted catenary is always cradled, the proportion observed being to give half the dip of the main cables and twice the splay. This is an empirical formula with no special theory to support it. The inverted catenary so placed is a parabolic wind tie in an inclined plane. The horizontal component is useful against wind pressure, and has already been discussed. The vertical component, valueless against wind pressure, is hardly more effective against undulations or vertical movement of the roadway. Like the main cables it is subject to deformation under partial live load and in exactly the same way, and can only cause additional and useless tension in the suspenders and main cables. Under a fall of temperature both catenaries will contract, and the tension produced in the suspenders and ties as well as in the cables themselves may become very great. On the other hand a rise in temperature will cause the wind tie to go slack, and so become inoperative.

Considered as a means of stiffening the roadway the inverted catenary must be regarded as ineffective and dangerous, and the consideration of this device illustrates very clearly the need to distinguish between the two causes of movement in the roadway as enunciated in the paragraph on "Wind ties and other devices" and the provisions of separate and independent appliances for preventing these two different kinds of movement.

Alternate Cable Suspension.

This device consists in having two distinct sets of main cables and in fastening the transoms to each set alternately. The method is described in Vol. XXI of Professional papers of the Corps of Royal Engineers, and was used on the Chamagarh Bridge over the Gilgit river, span 432 feet.

The theory is that the roadway being supported by two perfectly distinct pairs of cables, the undulations set up in one pair of cables by a live load on the bridge will be neutralised when the load is transferred at the next transom to the other pair of cables. The wave is thus damped almost as soon as generated by a following wave at half wave distance.

Clearly this device can only be used with unstiffened bridges. Since the undulations are communicated to the cables through the medium of the roadway we have conflicting undulations in that part of the structure which introduces somewhat of the elements of impact. The design is not so sound as constructing the roadway inherently rigid.

Stiffened Suspension Bridges.

As already pointed out these divide naturally into Type II (*a*) Bridges with stiffened roadways, and Type II (*b*) Bridges with stiffened cables.

Stiffened Roadway.

Type II (*a*).—There are a number of devices available for stiffening the roadway, the first device put into use was that of introducing strays by which various points on the roadway were fastened to the tops of the towers.

Stays.—Merriman and Jacoby's Roofs and Bridges, Part IV, give the following account:—

"All suspension structures built between 1810 and 1850 were of the Finlay Type, the roadway being hung from the cables by vertical rods; to prevent oscillations, however, inclined rods called stays were attached to the roadway at various points and carried to the tops of the towers, while guy rods were run laterally and downwards from the roadway and secured to points on the banks of the stream. In spite of these precautions these bridges were subject to violent oscillations in gales of wind, and many were destroyed. Even under the passage of ordinary traffic they were liable to great deflections, and it was then generally supposed that the system could not be advantageously adapted to rail road structures."

Merriman and Jacoby then enter into a mathematical investigation of the design of such stays showing that under live load stresses the behaviour of these members is not in consonance with the behaviour of the suspenders. They conclude:—

" It thus appears impossible rationally to design a stay to resist the stresses that come upon it. Stays moreover do not act in unison with the suspenders under changes of temperature, while as stiffeners their utility is far inferior to that of a stiffening truss. For all these reasons stays practically have been abandoned in recent designs for suspension bridges, and it is not likely that they will be hereafter used in structures where a truss is used."

The same remarks apply *mutatis mutandis* to the vertical components of simple wind ties.

In spite of these disadvantages the Brooklyn Suspension Bridge completed in 1883 with a central span of 1595 feet and side spans of 930 feet, and which is of this type, is still successfully carrying its traffic load of a foot walk, two wagon roadways, and two tracks for light rail road passenger traffic.

Stiffening truss.

This device was first used by J. A. Roebling in 1885 for the heavy railway bridge over the Niagara River. The object of the device is to suspend a truss from the main cable, the truss being so designed that a concentrated or partial load is distributed uniformly along the cable, thus obviating any deformation of the cable or roadway under partial loading.

This device is of such importance in the design of suspension bridges that we will consider briefly the theory on which it depends.

Unlike an ordinary truss, it is not the function of this truss to transfer loads to the supports at its ends, but to distribute loads uniformly to the suspenders, and so to the cables.

Since, however, the truss is capable of carrying a load, the combination of truss and cable is a redundant framework, and it is necessary to make certain assumptions before we can find a solution.

The assumptions usually made are that the cable retains its parabolic form and that the lengths of the hangers are so adjusted that the dead load is entirely carried by the cable.

Under dead load there is no stress in the truss which has to be designed for live load only.

If the cable remains a parabola, it follows that the stresses in the hungers are all equal, for the parabola is the curve of equilibrium only when the load is uniformly distributed.

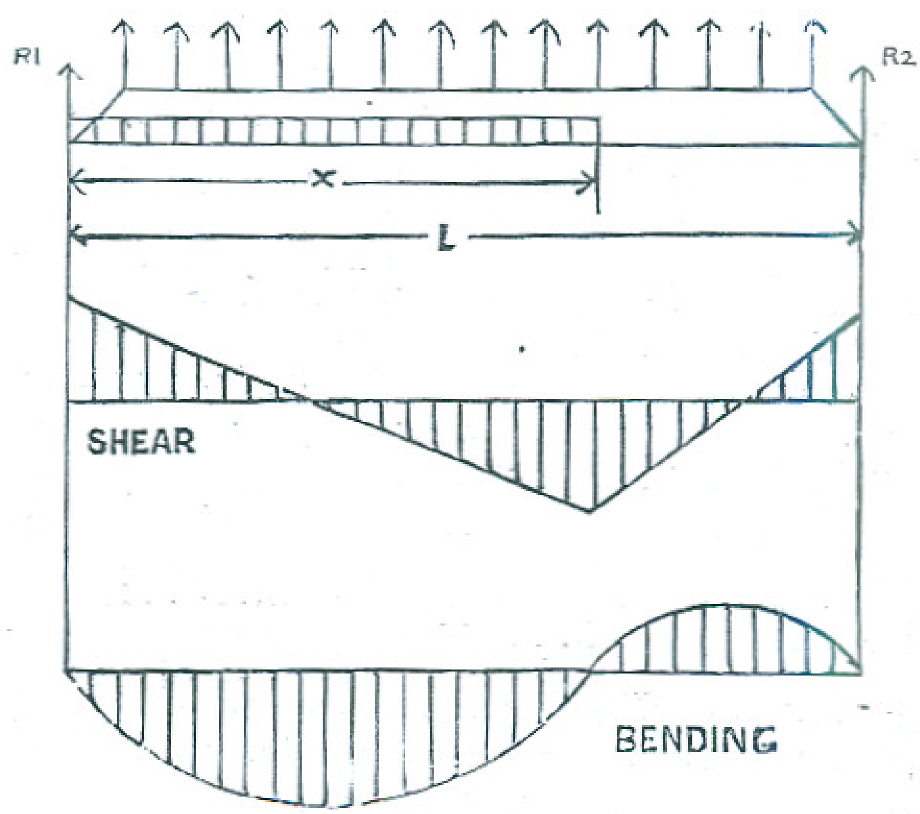


Fig. 5.

It is usual to fasten the truss to the abutments at each end so that the reactions at the ends may be either positive or negative, and to leave the truss free to expand longitudinally under changes of temperature. If the truss fully distributes the partial load to the cable, the sum of the reactions at the ends of the truss must be zero.

In Fig. 5—R_1 and R_2 are the reactions at the ends. L=the length of the truss. Let a live load w per linear unit extend out from the left support a distance x.

Let w^1 be the uniform upward pull of the hangers per linear unit so that the total upward pull of the cable on the truss is $w^1.L$.

Then the live total load$=wx$

and by assumption total live load on truss = total live load on cables

$$i.e.,\ wx = w^1L \quad \therefore\ w^1 = \frac{wx}{L}$$

Take moments about either end.

$$R_1 = +\frac{wx^2}{2}\left(1-\frac{x}{L}\right)$$

$$R_2 = -\frac{wx^2}{2}\left(1-\frac{x}{L}\right)$$

When $x = L$ or $x = O$ $\quad R_1$ and $R_2 = O$

$$R_1 = \frac{wx}{2} - \frac{wx^2}{2L}$$

$$\frac{dR_1}{dx} = \frac{w}{2} - \frac{2wx}{2l} \text{ equate to zero for max. value.}$$

$$x = \frac{L}{2}$$

Substituting this value

$$R_1 = \frac{1}{8}wL$$

Shear at any point P in the loaded segment distant y from R_1

$$= \frac{w}{2}\left(x - \frac{x^2}{L} + \frac{2xy}{L} - 2y\right)$$

which is zero when $y = \frac{x}{2}$ and $-R_1$ when $y = x$

thus the negative shear at the head of the load equals the reaction The unloaded segment may be calculated in the same way.

The Mff. occurs when the shearing force changes sign and is nil when the shear force is a max.

Mff. at any point in the loaded segment distant y from the left end has the value Mff. $= (R_1 y) + (w_1 y \times \frac{y}{2}) - (wy \times \frac{y}{2})$

Substitute $R_1 = \frac{wx}{2} - \frac{wx^2}{2L}$

$$w_1 = \frac{wx}{L}$$

$$\text{Mff.} = \frac{w}{2}\left(xy - \frac{x^2 y}{L} + \frac{xy^2}{L} - y^2 \right)$$

this becomes 0 when $y=0$ and has max. value; when $y = \frac{x}{2}$ substitute this value

$$\text{Mff.} = \frac{w}{2}\left(\frac{x^2}{4} - \frac{x^3}{4L}\right)$$

Differentiating and equating to zero $x = \frac{2}{3}L$

which gives the condition of loading for max. bending moment. Substituting this value

$$\text{Mfff} = \frac{wL^2}{54}.$$

Similarly the Mfff. for the unloaded segment occurs when one-third of the truss is loaded and equals $-\frac{wL^2}{54}$.

The max. bending moment for a self-supporting simple truss is $\frac{wL^2}{8}$ while for the suspended truss it is $\frac{wL_2}{54}$.

This is the common theory of the suspension truss as first evolved by Rankine. For a very heavy dead and a light live load it is not far from correct; for a light dead load and a heavy live load (the usual conditions of a light suspension bridge) the initial assumptions do not hold, and the stresses may be very different from those given above. The above theory will be found to break down in the particular case of a concentrated load at one end of the bridge, and takes no account of the stresses undoubtedly introduced into the truss by any variation of the dip of the main cable under live load or under variations of temperature.

Incomplete and unsatisfactory as the theory is, it has been reproduced in full in this paper because it illustrates very clearly one of the most important characteristics of suspended roadways, and as a corollary to this theory we may write down the condition :—

If the roadway of a suspension bridge is so constructed as to be capable of transmitting bending moments throughout its length, then it must be so designed as to be able to withstand the max, bending moment which the live load can produce, the term live load in this case including all loads other than dead weight of structure.

The converse of this condition has already been stated under the section on roadway of unstiffened bridges.

There is no compromise possible between the two types, a suspension bridge must be either flexible or have properly designed stiffening members.

It would be out of place in this paper to attempt any further elaboration of the theory of stiffening trusses, but certain definite advantages attach to the introduction of a hinge in the centre of the truss.

In designing these trusses it is usual to make the truss of uniform section throughout and to design the top and bottom booms to take either tension or compression.

P. W. D. Paper No. 51 states that the depth of the truss should not be less than $\frac{1}{24}$ th of the length. In actual practice on bridges of about 150 foot span the depth of the truss is usually 5 feet, and the transoms are placed 5 feet apart, which gives a square panel truss. The design commonly adopted being panels with cross bracing. This design is apt to prove unsatisfactory where the truss is a timber one ; timber contracts and expands under changes of humidity apart from temperature changes, and this often results in joints working loose and the truss losing much of its stiffness. Since stiffness or rigidity is the characteristic most desired in this member, the design should be selected with that end in view. A lattice girder, which it would be quite easy to construct in timber, would probably prove effective.

The horizontal stiffening truss against wind pressure, placed as it usually is in the plane of the transoms, does not lend itself readily to the lattice type of construction. There is, however, room for ingenuity in devising a form of roadway laid on top of the transoms which would also give the horizontal stiffness required. In an all metal design steel troughing might be made effective, and it would not be difficult to device such a floor of timber, though it would require a wearing surface to protect it from destruction by the constant passage of traffic. Not infrequently the lower booms of the vertical trusses serve also as booms to the horizontal truss, and in this case they require to be designed with sufficient strength for both services.

Stiffened Cables.

Many designers of suspension bridges have turned their attention to the origin of the deformations and have preferred to introduce stiffness or rigidity into the cable itself, so that the suspenders transmit the load direct to a structure which is inherently rigid. Figures 6, 7, 8 illustrate different types.

In figure 6 rigid cross bracing has been introduced connecting the cable and the roadway, and the structure has become an inverted steel arch. If a hinge be introduced at C, the design approximates very closely to a cantilever in which the projecting arms are balanced by the tension in the backstay. The upper chord need no longer be a parabola, and the lower chord is subject to longitudinal stresses; in fact the distinctive features of a suspension bridge have disappeared.

Figure 7 shows a type which is in effect an inverted steel arch with or without hinges according to the method of attachment of the cables to the tops of the towers. Both types 6 and 7 are generally used with cables built up of rigid eyebars so that they become rigid-pin-jointed structures, and the methods of solution applied to steel arches and cantilevers apply equally to them.

Figure 8 is an inverted three-hinged arch, and as such is statically determinated by the usual methods.

In this type the axis of the girders is first drawn as a parabola. The vertex of the parabola so drawn is joined by straight lines to the points of support, and the lower curve, is found by setting off a vertical distance below the original parabola equal to the height of the straight line above it.

Military Engineering, Part III B, gives a detailed description of the construction of these cable girders for a light suspension bridge using flexible wire rope. It is necessary in this case to manufacture the girders on the ground before erecting them, and in order that the stresses when in use shall correspond with those calculated, it is essential during manufacture to introduce stresses approximating to the working stresses. This is not easy to do with unskilled labour, and probably accounts for the system being very seldom adopted for light bridge work. When the cables are made of rigid eyebars and the centre hinge is a genuine pin joint, the cables can be put together *in situ*.

When the cables are stiffened and not the roadway, the most suitable place for horizontal stiffening against wind pressure is in the plane of the cables.

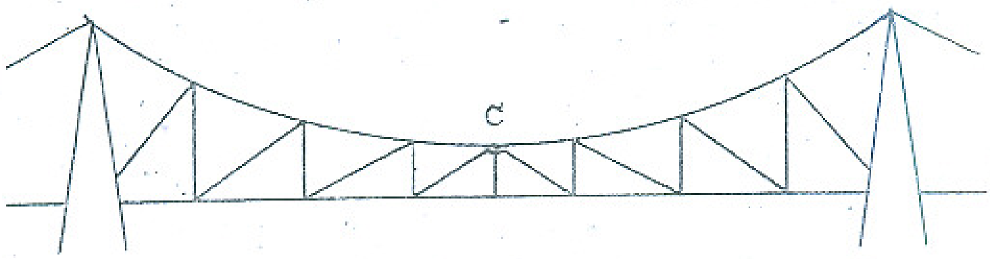


FIG 6

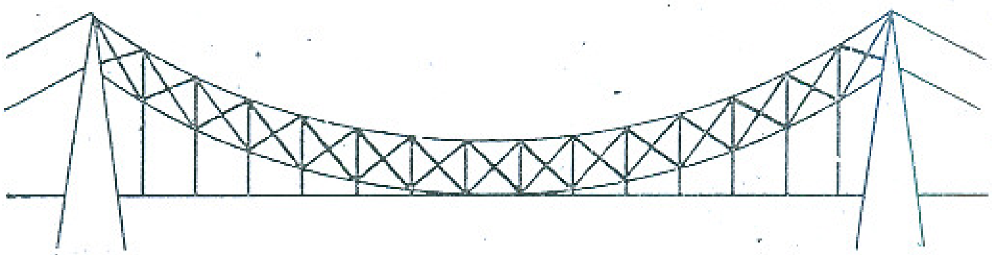


FIG 7

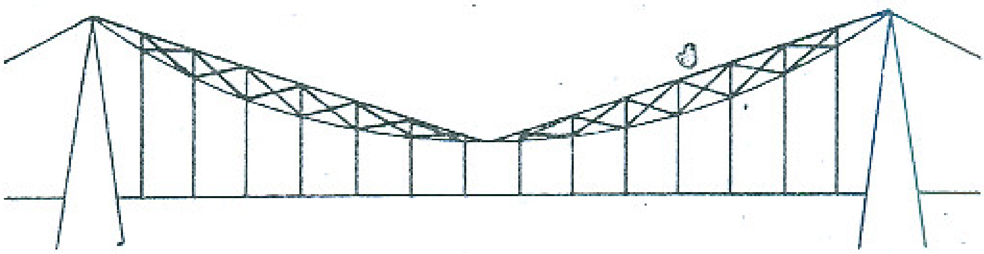
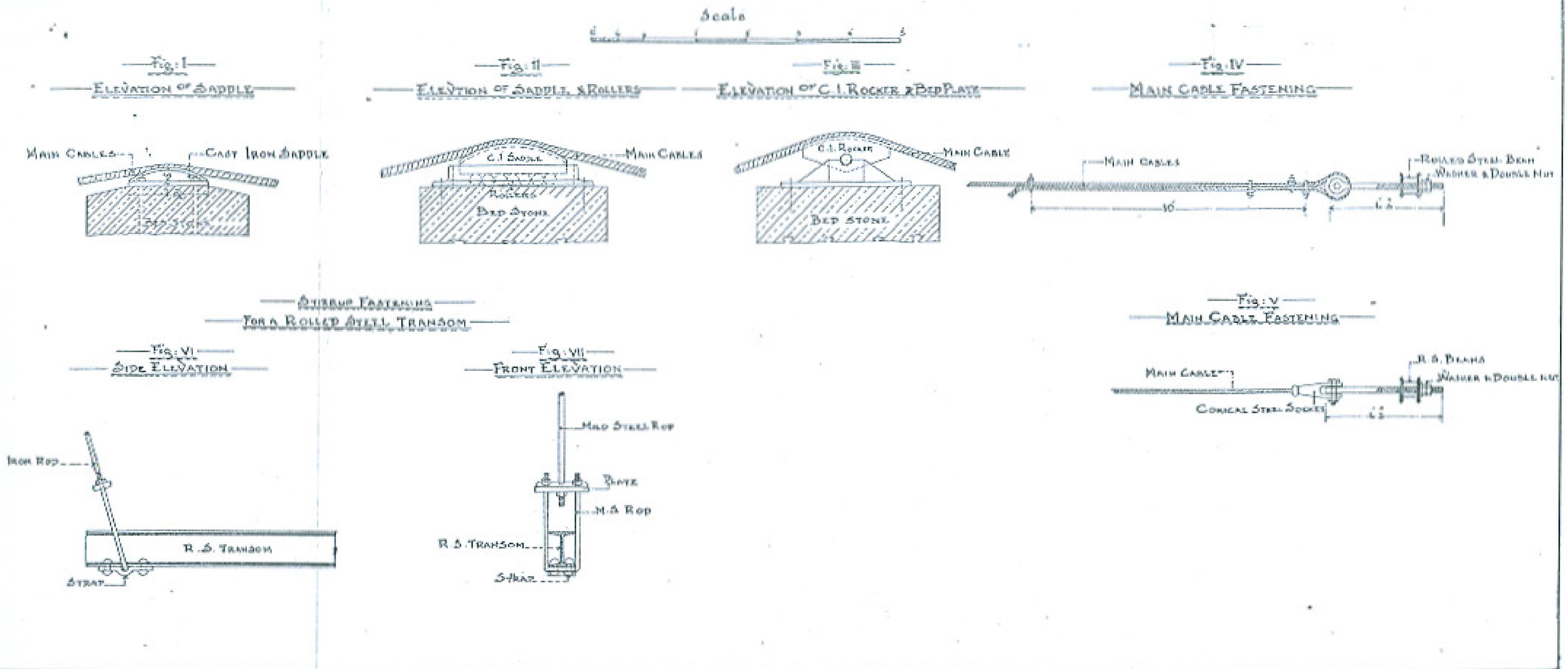


FIG 8

PLATE 1

THE DESIGN & CONSTRUCTION OF LIGHT SUSPENSION BRIDGES



APPENDIX B.

Name of river.	Name of bridge.	Span.	Width of road.	Type.
..	Vishnuprayag	185′	6′	Cables cradled. Roadway stiffed. Girder with central hinge.
Indus ..	Purlap ..	337′	10′	Cable cradled.
Astor ..	Ramghat ..	171′	7′	
Astor ..	Garikot ..	{ 162′ 81′ }	7′	
..	Olousaa ..	{ 246′ 123′ }	8′ 6″	Steel piers.
Hanga ..	Tashot ..	304′	..	Cables cradled.
Gilgit ..	Gilgit ..	{ 368′ 152′ }	5′	Inverted catenary windties.
Gilgit ..	Chamogarh	432′	..	
Spiti ..	Dankar. ..	146′	3′	Cables cradled. Road unstiffened. Windties horizontal.
Sutlej ..	Poo ..	174′	3′ 3″	Cables cradled. Road unstiffened. Windties horizontal.
Sutlej ..	Sholtu ..	141′	3′ 3″	Cables cradled. Road stiffened. Continuous girder.
Sutlej ..	Wangtu ..	150′	6′	Steel towers. Road stiffened. Continuous girder.
Sutlej ..	Chaura ..	159′ 4″	6′	Cables cradled. Road unstiffened.
Sutlej ..	Jakri ..	140′	3′ 3″	Cables cradled. Road stiffened. Continuous girder.
Sutlej ..	Rampur ..	168′	5′ 8″	Road stiffened. Continuous girder. Windties horizontal.
Sutlej ..	Luri ..	175′	6′	Cables cradled. Road stiffened. Continuous girder.
Sutlej ..	Suni ..	133′	6′	Cables cradled. Road stiffened. Continuous girder. Horizontal stiffening. Stays and horizontal and inclined windties.
Sutlej ..	Dehar ..	230′	7′ 6″	Cables cradled. Road stiffened. Steel hinged girder. Horizontal stiffening.
Nogli ..	Nogli ..	125′	5′ 8″	Road stiffened. Continuous girder. Horizontal stiffening and horizontal windties.

Discussions.

Mr. A. St. G. Lyster introducing his paper said that it was with some diffidence that he offered for their criticism this paper on light suspension bridges.

He regretted that the paper was neither a description of some notable work completed and that it did not advance any new theory.

An inspection of the suspension bridges in the Upper Sutlej Valley, and an examination of the calculations made when they were designed, convinced him that in some cases the fundamental principles governing the design of these structures had been lost sight of and that the true functions of the various parts of the structure had not been fully appreciated.

There existed already a number of departmental papers on the subject but these appeared to him to be inadequate as guides to a scientifically correct design. Either they described a single suspension bridge which had been erected, thus giving but one solution of one particular problem or as in P. W. D. paper No. 51 they gave a standard type without pausing to consider the relative merits of possible variations in the design.

In this paper therefore he endeavoured to give prominence to the principles which governed the design of suspension bridges, and in examining each part of the structure in turn to show the relative merits and demerits of different designs and the effects that might be produced in other parts of the structure.

No useful purpose would have been served by inflating the paper with long transcriptions from text books. Accordingly the calculations in the paper had been reduced to a simple statement of results, the object being to give some guide to the magnitude of the stresses involved.

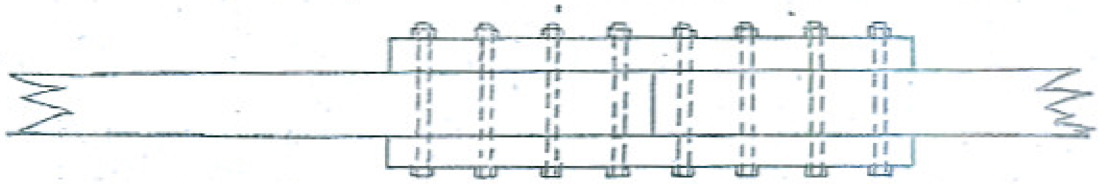
There were one or two corrections which he wished to make. On page (6) a formula for the weight of the cable was given based on an approximate formula for the length of the cable $=2\sqrt{\left(\frac{S}{2}\right)^2+\frac{4}{3}D^2}$.

This formula was taken from P. W. D. paper No. 51. A more accurate and more generally used formula though still only approximate was length of cable$=S\left(1+\frac{8}{3}\left(\frac{D}{S}\right)^2-\frac{32}{5}\left(\frac{D}{S}\right)^4+\ldots\right)$ of which it was generally sufficient to use the first two terms only. This latter formula gave results about half per cent. higher than the former one.

On page 7 the statement that "the change in Dip will be negligible" should be corrected to "the change in Dip will be 0·22 feet."

Mr. J. H. Johnston referred to one or two factors, not mentioned by the author, which he considered controlled the design of stiffening girders.

In the Punjab the girders were generally of timber. Both booms were subject to tension and the joints had to be designed accordingly. There were only two ways of making a timber tension joint, the plain fish plate joint and the tabled fish plate joint. In the first the load was transmitted through the bolts and in the second through the tables or keys.



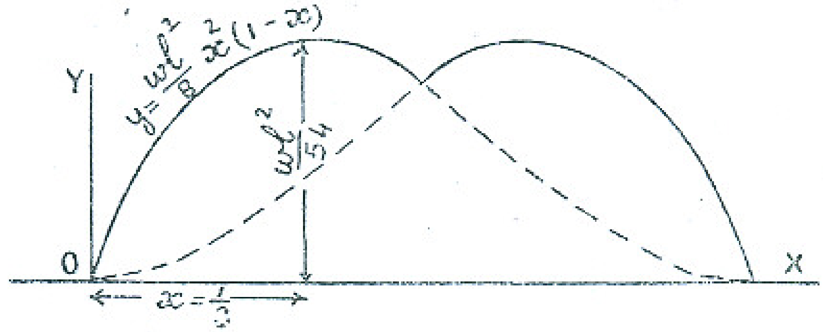
Plain Fish Plate Joint.

The tabled joint requiring better workmanship was not suitable for the Punjab. In the plain fish plate joint the number of bolts necessary depended upon the safe compressive stress of the timber used, provided the bolts were strong enough to resist bending. If the safe compressive and tensile stresses of the timber were equal, then for each inch in the effective depth of the beam one inch of bolt diameter was required. Two 10″×5″ sleepers connected by $2\frac{1}{2}$″ fish plates would require 8 one-inch bolts on each side of the joint if the effective depth of the beam were taken as 8 inches. It was therefore essential to use as few joints in the booms and to obtain as long timbers as possible. In other countries timbers up to 60 feet in length were used in timber girders but these could not be had in the Punjab. For the Pandoh suspension bridge span 230-feet 12 foot sleepers were the largest sizes available and 3,000 bolts were required for the four booms.

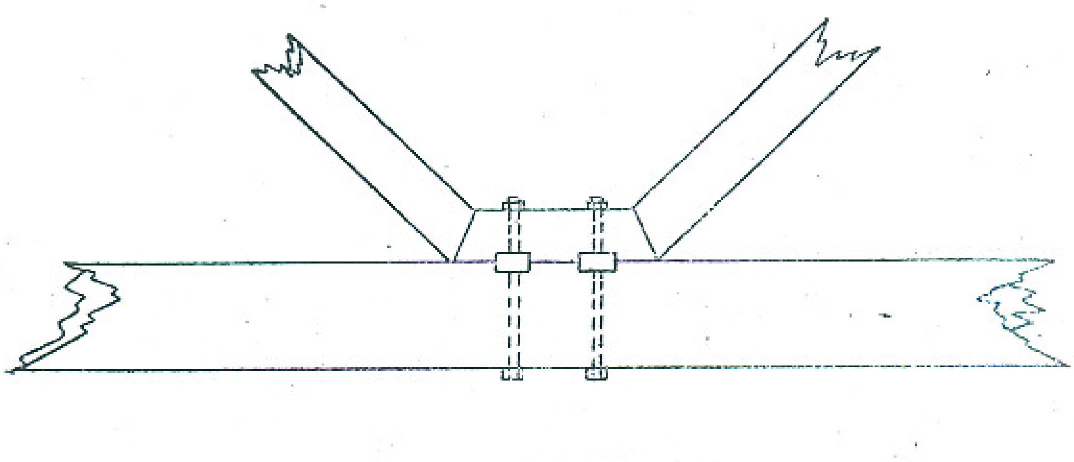
In one or two bridges in the hills the joints in the booms had been designed to carry only a small proportion of the safe load in the timber. The bottom boom of the Victoria Bridge at Mandi was 10″×12″ capable of carrying a load of 50 tons. The joint was made with three $\frac{3}{4}$″ bolts and would fail under a load of 5 tons. In the Oot Behali Bridge near Largi also the efficiency of the joints was about 5%. This was his explanation of what Mr. Lyster put down to the action of humidity on page 21. Although the girders of the Victoria Bridge, even when new, could not take the full load of a stiffening girder, as ordinarily designed, the bridge was still wonderfully stiff. The explanation for this stiffness was he thought due to the resistance to bending of the booms and of a large 12″×12″ timber fixed under the floor beams in connection with the lateral bracing. The bridge would be just as stiff if the diagonals were removed and both booms bolted to the floor beams. Mr. Lyster's remark on page 21 "there is no compromise.............." therefore was incorrect. He maintained that between the limits of maximum deflection in the unstiffened bridge and minimum deflection in the stiffened one it was possible to design stiffening members for any desired maximum deflection provided such members were sufficiently flexible to allow of such deflection without failing.

Since the booms used up so many bolts it was desirable to reduce the latter wherever possible. The number at any point should be proportional to the ordinate of the maximum bending movement diagram. This diagram for a girder without centre hinge under a uniform rolling load was not often shown in text books. It consisted of two cubics of the types $y = \frac{wl^2}{8} x^2(1-x)$

$$\text{where } x = \frac{\text{distance from either abutment}}{\text{Span.}}$$



It was common to design the diagonals for compression only. In this case the load was generally transferred to the booms through cleats and provision had to be made for horizontal shear between the cleat and the boom. In the Pandoh Bridge this shear was about 12 tons and was being taken up by two hard wood keys and three 5/8″ bolts.



There were two methods of bracing the top booms against lateral movement—

(1) side bracing by diagonals from the ends of the floor beams,

(2) overhead bracing.

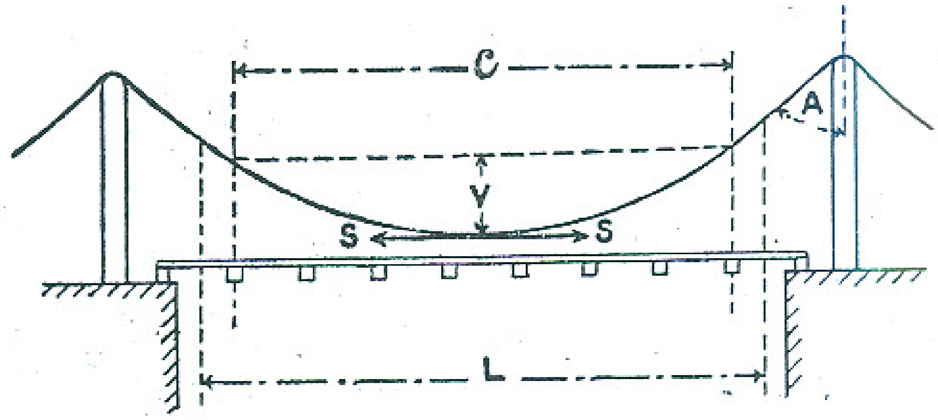
The cost of the bridge might depend largely on which method was adopted. The first method was simpler but the cables could not conveniently be brought below the level of the top boom without fouling the bracing. This meant higher towers, longer cables and longer floor beams, but if the approach road went under the cables it gave an easier curve to the road. It might be economical to use overhead bracing in the centre of the bridge where the cable was below the level of the top boom and side bracing towards the abutments.

He asked if Mr. Lyster could explain why the girders were never placed under the roadway. If this were done and the cable were fixed at the level of the lower boom the following advantages would be gained:—

1. Reduction in height of towers.
2. Shorter cables.
3. Lighter floor beams, the girders being economically spaced
4. Simpler and cheaper lateral and cross bracing.
5. Shorter suspenders.

The stiffening girder was designed on the assumption that under no load there was no reaction on the abutments. This was a point which should be remembered on inspecting a bridge as he almost always found a considerable upward reaction and the nearest suspenders quite slack.

Rai Bahadur Makhan Lal said that it was wrong to take span equal to the horizontal distance between points of support of cable as was done by the author. It was really equal to the horizontal distance between the points where the first transoms next to the abutments were hung, as the actual parabola was formed between those points and not beyond them. He also said that in calculation, L should be taken equal to span plus length of bay between two transoms as shown on the sketch given below and not the distance between towers.



The formulae commonly used for the calculation of S and T were $S=\frac{(W_1+W_2)\ LC}{2\times 8V}$ $T=S\sqrt{1+\left(\frac{4V}{C}\right)^2}=S\ \text{cosec}\ A$

where S=Tension in cable at centre

T=Tension in cable at piers

W_1=Live load per running foot

W =Dead " " " "

L=Length of bridge loaded on cable

C=Span of Parabola.

V=Versed sine of curve, *i. e.*, deflection in feet

W=Total load on each main cable $L\left(\frac{W_1+W_2}{2}\right)$

A=Angle of cable at piers

V varied between $\frac{C}{10}$ and $\frac{C}{15}$; the larger V was, the less was stress in the cable, but higher the piers and more unsteady the bridge. It would generally be found that the most economical value for V was $\frac{C}{14}$ and if distance between transoms was made 7 feet, calculations were much simplified.

A splay of 1 in 40 was generally given to the cables with the object of steadying the bridge against lateral oscillation and effect of wind.

It was essential to design bridges for mean temperature of the locality where it was to be erected and to erect it at the same temperature as far as possible, otherwise there was danger of great initial stresses taking place in the stiffening girder.

Area of a steel wire rope made up of several strands of wires was much less than that of a steel rod of the same circumference.

It was advisable to make cables of ropes of all equal diameters, the number of which was seven or multiple of it, as they made a perfectly homogeneous and circular cable.

On each side of the bridge it was of considerable importance not to have more than three cables as it was then difficult to distribute loads from the slings equally on each cable.

The object of camber in a suspension bridge was first to allow for deflection and secondly for the optical illusion given by a straight horizontal line.

Rai Bahadur Lala Canga Ram suggested to the author that suspension bridges with cables of steel links would be stiffer than those with cables of wire rope, and he requested the author to furnish to the Congress a second paper dealing with steel link cables.

MR. A. S. MONTGOMERY said that Sardar Bahadur Lehna Singh had said that wood work and iron work should be very carefully executed but this was almost impossible in the case of out-of-the-way places, for example the Dankar Bridge in Spiti. Allowances must be made for rough work. The idea of working to special temperatures was an impracticable refinement. Engineers should leave to the makers the number of wires per cable and not attempt to prescribe this for them. Although Pandol Bridge was on the Beas River it was impossible to obtain more than 12′ sleepers. In America suspension bridges were sometimes roofed to protect from snow. This example might be copied by them in places where such protection was necessary and life of bridge might be thus increased.

MR. SANGSTER said that this was a paper which he had at first thought would not perhaps lend itself much to discussion as it was limited to structures in the hills, where unfortunately very few of them had had the luck to serve. These light suspension bridges, as Mr. Lyster pointed out, were very useful in the hills when the depth and velocity of the water in the river rendered the erection of centring impossible. Had the depth and velocity of water in our canals prevented them from using centring they would no doubt have adopted them. It was a very interesting paper and they were indebted to Mr. Lyster for bringing it before them.

MR. LYSTER in replying to the discussion said that Mr Johnston had made a valuable contribution to the subject by drawing attention to the practical difficulties of constructing a tension joint in a timber beam which would develop the full strength of the beam. This might indicate the principal cause of inefficiency in wooden trusses; though shrinkage of the timber was always liable to cause trouble. In the case quoted by Mr. Johnston a beam 15″ × 10″ in section was built up of sleepers 10″ × 5″ in section and there was hardly room for the number of bolts required. It would appear *prima facie* that an unsuitable material was being used and it might be remarked that while the relative strength of mild steel to timber was 160 to 7 the relative weight for members of equal strength was steel 33 to timber 64. Such a reduction in the dead load would effect an appreciable saving in the main cables. Assuming that special circumstances precluded the use of steel in the trusses, he believed the use of glue for joining wood surfaces had not received sufficient attention. For propellors and other parts of aeroplanes glue was used with a tensile strength of 1,100 to 1,350 lbs. per square inch which was amply sufficient to develop the full strength of the timber.

The comparative stiffness of the Victoria Bridge noticed by Mr. Johnston was probably due to the dead load being large compared to the live load, the most inefficiently designed girder would help to stiffen a bridge by increasing the ratio of dead to live load.

With regard to stiffening the top booms; the diagonal bracing to the extended transom was used on all the Upper Sutlej Bridges and no difficulty was experienced in bringing the main cables down to within seven or eight inches of the centre transom, this strutting was a precaution against the girder overturning, the overhead bracing was better but was more expensive. This arrangement of the cables nearly fulfilled that proposed by Mr. Johnston; it would be possible to place the centre transom above the cables but this would introduce difficulties in erection; if the road was placed above the stiffening girders it was necessary to duplicate the transoms and add hand rails and the dead weight was increased.

As regards Mr. Johnston's proposal to use what the author would describe as a flexible stiffening girder making a compromise between a stiffened and unstiffened bridge, he would like to see such a design very much indeed. He did not think that it was practicable.

Sardar Bahadur Lehna Singh had drawn attention to the likelihood of failure by shear between the bolt hole and the edge of the timber in a tension joint. This was known as failure by crushing in steel rivetted joints and was always allowed for in the design. The author agreed with him on the need for proper design of each joint and its iron work in timber trusses.

Referring to the remarks made by Rai Bahadur Lala Makhan Lal the author said that the assumptions made in the theory of the stiffening truss were given on page 18 of the paper. Cradling had been dealt with at some length in the paper stress being laid on the increased tensions caused in the suspenders and main cables. As regards the selection of a suitable cable this was best left to the makers who used special grades of steel for such work and who were quite prepared to guarantee the strength and quality of the ropes supplied. Camber was recommended in some books only on the grounds of appearance, it must be remembered that the height of the towers increased by the amount of camber given.

In reply to Rai Bahadur Lala Ganga Ram the author said that steel link cables had been extensively used in the past for large suspension bridges. Later steel wire rope came into favour because it could be manufactured with a higher tensile strength than steel bars. Wire ropes were lighter and easier to transport and erect. Steel links would offer a certain amount of resistance to deflection by friction at the pin joints.

The author added that Mr. Montgomery was quite right in pointing out that light suspension bridges were frequently built in places so inaccessible that even when a certain amount of skilled labour was imported it was almost impossible to do good joinery. Dankar in Spiti was reached by traversing a glacier and then marching through country so barren that one had to carry one's own fuel. Mr. Montgomery had replied to several of the points raised by other speakers. With regard to coal tar as a protective coat, he was afraid that in some of these out of the way places the annual coat of coal tar did not always reach the bridge. The plan of roofing a suspension bridge to protect it from weather had not, as far as he knew, been tried in this country. The average life of deodar timber in bridges in the Himalayas was usually taken as thirty years.

Referring to the correction proposed in calculating span, a rope hanging under its own weight took up the curve known as a catenary. When carrying a uniform load very much greater than its own weight the curve became a parabola passing through the points of support. In the case of a suspension bridge the points of attachment of the suspenders lay on the parabola and the wire rope took up a straight line position between them. The long gap between the points of support and the point of attachment of the first suspender would cause some distortion of the curve but would not have any appreciable effect on the strength of the rope, if the load were correctly calculated. The correction proposed by the speaker in calculating the length of the road was sound.

It is not clear what simplification was introduced into the calculations by the use of a factor of 7 in selecting the Dip ratio and length of bay.