March 7, 1994

Dr. Joel I. Abrams
International Bridge Conference
Engineers' Society of Western Pennsylvania
337 Fourth Avenue
Pittsburgh, PA 15222

Dear Dr. Abrams:

Enclosed is a paper titled "Cable-Net Bridge System" by our graduate student, Swaminathan Krishnan. This paper is being sponsored by our department for your Student Paper Conest of the 11th Annual International Bridge Conference.

Sincerely,

[Signature]

John E. Merwin
Professor and Chair
CALL FOR STUDENT PAPERS

The 11th Annual International Bridge Conference (IBC) is proud to sponsor a Student Paper Competition. Student papers are being solicited from students at every college and university within the United States offering a Civil Engineering major. The competition is open to all graduate and undergraduate students. Papers will be reviewed by the student’s department chairman, and one finalist will be submitted to the IBC Student Awards Committee.

The author of the winning paper selected by the Student Awards Committee of the IBC will receive a $1,000 Grant Fellowship from the Conference, along with complimentary registration, hotel accommodations and travel to the 1994 International Bridge Conference.

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STUDENT PAPER COMPETITION

INFORMATION SHEET

Complete Information Sheet and return to your Department Chairman with your paper.

TITLE OF PROPOSED PAPER

CABLE-NET BRIDGE SYSTEM

__________________________________________________

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Joel I. Abrams
International Bridge Conference
Engineers' Society of Western Pennsylvania
337 Fourth Avenue
Pittsburgh, PA 15222
IBC Phone Number: 412/261-0710
Cable-Net Bridge System

Swaminathan Krishnan

1. Abstract

It is well-established that cable-stayed bridges are suitable for main spans in the range of 250m-500m and cable-suspended bridges for main spans beyond 1500m. In the intermediate span range of 500m-1500m, both systems are deficient and there is need for a better option. Using the fundamental concept of ‘cable beams’ applied in suspended roof structures, the author proposes a new ‘cable-net system’. Detailed designs of this system and a cable-stayed bridge system are carried out for a main span of 800m. By means of a comprehensive comparative analysis of bending moments, axial forces, deflections and amount of steel used, the paper demonstrates that the cable-net bridge system provides an economically, structurally and aesthetically viable alternative to the cable-stayed and cable-suspended bridge systems for intermediate spans.

2. Introduction

The need for long span bridges may arise due to high foundation cost resulting from wide flood zones, channel instability, deep gorges etc. Cable supported bridge systems are distinguished from conventional bridge systems by their ability to span such wide crossings efficiently. Two types of cable supported systems are commonly used - cable-stayed bridges and cable-suspended bridges.

Cable-stayed bridges are suitable for the main span range of 250m-500m. A large number of bridges have been successfully completed and are in use. However, as the span increases, cable-stayed bridges become uneconomical due to the following reasons:

1. The cable angles decrease and hence vertical load carrying efficiency reduces.
2. Cable sag which is proportional to the square of the cable span becomes large resulting in cable inefficiency.
3. Taller towers are required to counter the above problems which in turn leads to large slenderness ratios for the towers and consequent buckling problems.

Cable-suspended or suspension bridges are commonly used for long main spans of 1500m or more. While suspension bridges with central span length of about 2000m have been completed and those with lengths ranging between 3000m and 5000m are under various stages of design and development, systems of this type have very low stiffness resulting in large deflections. In addition to poor performance with regard to serviceability, high laying and erection costs are inherent problems associated with this system.

Intermediate spans ranging between 500 and 1500m correspond to the upper limit of cable-stayed systems and the lower limit of suspension systems. Even though cable-stayed bridges are superior to suspension bridges with regard to serviceability, ease of erection and economy of construction, both systems have their deficiencies and, to date, there is no viable alternative.

In this study, the author proposes the use of a pretensioned bridge system for spans in the

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range of 500-1500m. This system, best-described as a \textit{cable-net bridge system}, shall be shown to combine the advantages of both cable-stayed and suspension systems. The primary objectives of the study are:

- Propose the structural form and configuration of cable-net bridge system.
- Propose a rational method for determining the optimal amount of cable pretensioning based on the flexibility approach.
- Design a cable-stayed bridge and a cable-net bridge for a main span of 800m.
- Compare the two bridge systems in terms of bending moment, axial forces and deflections and study the cost-effectiveness in terms of amounts of steel used.

3. System Proposed

Fig. 1a shows the typical form of a suspension bridge system. It is characterized by a long suspension cable spanning the whole length between the two towers. Vertical hangers transfer the load from the deck to the suspension cable and on to the towers. Stiffer systems than the suspension system can be achieved if a second set of cables with reverse curvature is connected to the suspension cable, thus forming a \textit{cable beam}\textsuperscript{2}. Using this principle, an efficient bridge system, the cable-net bridge system, which retains all the useful features of the suspension system, is proposed. Fig. 1c shows the essential features of this system. For a span of 800m, the cable-net bridge system is shown to be superior to the cable-stayed bridge system (Fig. 1b) which, in turn, is considered better than the suspension bridge system in this span range.

3.2.1. Structural Action

In the cable-net bridge system, the higher the stiffness of the cable-net, the better will be the behavior of the deck in bending. Cable beams will be quite stiff if tensioned to a level which ensures that both the suspension and prestressing cables remain in tension under any combination of applied loading. This is done by tying the two cables with diagonal ties (Fig. 1c). Furthermore, the two pylons are pulled back using the back-stays and anchored to the supporting abutments. The suspension cable which is pretensioned against the prestressing cable with the help of the diagonal ties, acts as the main load carrying member. In the main span, the load from the deck is taken by the vertical hangers which transfer it to the suspension cable through the ties. The suspension cable then transfers the load on to the top of the pylons which take it safely to the ground, through the foundation. The ties thus have a double function - that of prestressing and of load transfer. For the side spans, the load is directly transmitted from the deck to the pylons by inclined cable-stays.

Thus, for the cable-net bridge system, a better performance level is expected on the following lines:

a) The load transferring members, the hangers, being vertical are more efficient in their transfer than the inclined stays of the cable-stayed bridge system. Thus, even at the centre of the main span, cable areas are small.

b) The cable-net bridge overcomes the problem of shallow slope of interior cables in the cable-stayed bridges for this span range. Due to this, the height of the tower can be smaller compared to cable-stayed bridge towers.

c) The bending moment in the deck can be reduced compared to cable-stayed bridges since the deck is not used to create pretensions in the cables unlike in the cable-stayed bridges.
d) The cable-net is stiffer than both the stay cable and the suspension cable configurations and thus bending moment and deflection in the deck due to partial live loads are considerably reduced compared to cable-stayed and cable-suspended bridges. Hence, the height of tower in the cable-net bridge can be smaller than that of even cable-suspended bridges.

4. Analysis and Design of Cable-Stayed Bridge

4.1 Configuration

In the design of bridge systems, the decisions regarding configuration are very important and are usually arrived at by designers through experience and a clear understanding of the behavior of the system under various loading conditions. Some of the important parameters involved are: 1. Side span. 2. Height of the tower. 3. Pylon type. 4. Cable configuration - transverse and longitudinal. 5. Cable spacing along the deck. 6. Cable spacing along the tower. 7. Central panel length in the main span. 8. Basic section types of the various elements. Based on parametric studies by Gimsing\(^9\) and a survey of several existing bridge designs by the author, a nearly optimal configuration is generated and has the following features:

1. Pylons are chosen to be A-frames.
2. Based on the recommendations made in Ref. 6, for a main span of 800m,
   a) Side span = 0.31*Main span = 250m.
   b) Height of the tower above deck = 0.16*Main span = 130m.
   c) Distance from the top of the tower upto which cables are spread = 0.03*Main span = 24m.
   d) Multicable-stay system is chosen. Centre-to-centre distance of the cables along the deck = 10m.
   e) Central panel length in the main span = 60m.
   f) Box-sections are used for the pylons and the deck for increased torsional stiffness.
   Other details include the following:
      - the top of the deck is 30m above water level.
      - no.of lanes = 6.
      - width of each lane = 3.5m.
      - width of median = 1.0m.
      - width of footpath on either side = 1m.
      - total width of roadway = 24m.

4.2 Materials Used

1. Pylons: High Strength Steel, Fe360 with ultimate capacity of 360MPa.
2. Cables: High Tensile Steel of grade Fe1900 with UTS of 1900MPa.

4.3 Loading

The loads considered herein are according to Indian Road Congress Standards: 1. Dead Load. 2. Superimposed dead load. 3. Live load. The live load is considered to act as UDL spread over the area of the deck. Eight such load cases are considered (Fig. 2). Wind load is not considered since the torsional effect of the wind is expected to cause both systems to behave similarly and the objective here is to do a comparative analysis rather than a thorough analysis of the cable-net and cable-stayed bridge systems.
4.4 Optimal Pretensioning of Cables

Pretensioning the cables is perhaps the best way of reducing the bending moments in the girder. The flexibility method is used herein to arrive at the magnitude of pretension in each cable. The bridge is divided into twelve zones, each zone comprising of ten sets of cables. Due to symmetry, only one half of the bridge, comprising of 6 zones, is considered. For simplicity, only the pretensioning forces in the central cable of each zone (P₁ to P₆) are taken as the unknowns. The optimal values of these forces are evaluated such that the bridge hogs when it is loaded with dead load alone and it sags when the live load is included, i.e., the moments due to pretensioning balance the average of the moment due to DL and the moment due to DL+LL.

The flexibility equations are formulated as follows:

\[ m_{11}P₁ + m_{12}P₂ + \ldots + m_{16}P₆ + \frac{M_{DL}^1 + M_{LL}^1}{2} = 0 \]

\[ m_{21}P₁ + m_{22}P₂ + \ldots + m_{26}P₆ + \frac{M_{DL}^2 + M_{LL}^2}{2} = 0 \]

\[ \ldots \]

\[ m_{61}P₁ + m_{62}P₂ + \ldots + m_{66}P₆ + \frac{M_{DL}^6 + M_{LL}^6}{2} = 0 \]  \hspace{1cm} (1)

where \( m_{ij} \) stands for the moment at the mid-point of zone ‘i’, due to unit force at each of the cables of zone ‘j’ and \( M_{DL}^i \) denotes the moment at the mid-point of zone ‘i’ due to DL alone and \( M_{DL+LL}^i \) denotes the moment at the mid-point of zone ‘i’ due to both DL and LL on the structure without any pretensioning. The above set of equations aim to balance the bending moment due to half of combined dead and live loads by the pretension in the cables so that the design bending moments in the deck are uniform for various load cases. The prestressing forces, \( P₁ \) to \( P₆ \), are evaluated by solving these linear equations. A smooth curve is then drawn to obtain the pretensioning forces in all the other cables.

4.5 Analysis

Although the structure is geometrically symmetric about both the longitudinal and the transverse axes, only the longitudinal symmetry is taken advantage of, as the structure is analyzed for asymmetrically placed live loading conditions. Structural Analysis Program (SAP) has been used for analysis. The cables are modelled as truss elements under tension. The deck girder is modelled using plate elements while the pylons are modelled as beam-column elements. Hinged support conditions are applied at the base of the pylons (at the water level) and on both ends of the main girder. Hence, no moments are transferred to the foundation. The axial compression that is transferred to the load-bearing strata is nearly the same for both systems. As a result, only the superstructure design is taken up and the foundation analysis and design are left out.
4.6 Design Methodology

The working stress method is adopted for the design. Code of practice for Structural Steel design, *Bureau of Indian Standards*\(^5\), IS800-1984, is used. The design procedure adopted for the three types of elements in the bridge system is as follows:

Cables

Cables are direct tension members. The area required for each cable is determined by dividing the maximum force in the cable for all load cases by the allowable stress = 864 MPa.

Pylons

Pylons are made up of a number of beam-column elements. The codal provision for the design of these elements utilizes the expression for combined stresses given in Section 7.1 of IS800-1984 and is as follows:

*Combined axial compression and bending:* Members subjected to axial compression and bending shall be proportioned to satisfy the following requirement

\[
\frac{\sigma_{ac,cal}}{\sigma_{ac}} + \frac{c_m \sigma_{bcx,cal}}{\sigma_{bcx}} + \frac{c_m \sigma_{bcy,cal}}{\sigma_{bcy}} \leq 1.0
\]  

(2)

However, if the ratio \(\frac{\sigma_{ac,cal}}{\sigma_{ac}}\) is less than 0.15, the following expression may be used in lieu of the above

\[
\frac{\sigma_{ac,cal}}{\sigma_{ac}} + \frac{\sigma_{bcx,cal}}{\sigma_{bcx}} + \frac{\sigma_{bcy,cal}}{\sigma_{bcy}} \leq 1.0
\]  

(3)

The values of \(\sigma_{bcx}\) and \(\sigma_{bcy}\) to be used in the above formulae shall each be less than the maximum permissible stresses \(\sigma_{bc}\) for bending about the appropriate axis. In the above formulae,

\(\sigma_{ac,cal}\) = calculated average axial compressive stress.

\(\sigma_{bcx,cal}\) = calculated bending stress in extreme fibre.

\(\sigma_{ac}\) = permissible axial compressive stress in the member subject to axial compressive load only.

\(f_{cc}\) = elastic critical stress in compression = \(\frac{\pi^2E}{\lambda^2}\).

\(\lambda = 1/\ell\) = slenderness ratio in the plane of bending.

\(c_m\) = a coefficient whose value is 1.0 for the structure under consideration.

A trial section is assumed for the pylon. It is divided into 6 zones as shown in Fig. 4. For each zone, the combined stresses expression is evaluated for two cases - 1. the maximum axial compression and the corresponding bending moment and 2. the maximum bending moment and the corresponding axial compression. This is done for all the eight load cases and the maximum value of the expression for all cases is compared with unity. Based on this, changes are made to the assumed section and an iterative approach is resorted to until a satisfactory design is obtained.

Since the plates comprising the box girder of the pylon are laterally unsupported over a large length and width, stiffeners have to be provided to avoid local buckling of the plates. The moment of inertia required for a stiffener is given by,

\[
I = 3.66t^4 \left( \frac{w}{t} \right)^2 - \frac{281200}{f_y}
\]  

(4)

where,

\(t\) = thickness of the plate.
w = laterally unsupported width.

\( f_y \) = yield stress of the material of the plate in kg/cm².

Assuming an appropriate width-to-thickness ratio (less than 30 to avoid local buckling of plate between stiffeners), the moment of inertia required for each stiffener is calculated and the stiffener design is completed.

**Deck - box girder**

Based on the preliminary analysis, two key observations regarding the behavior of the deck girder can be made: (1) The axial compression on the deck is negligible. (2) \( M_{yy} \) is the governing bending moment.

This simplifies the design to a one-way bending problem. The check to be done is then obtained from eqns. (2) and (3) to be:

\[
\frac{\sigma_{bcy,\text{cal}}}{\sigma_{bcy}} \leq 1.0
\]  

(5)

\( \sigma_{bcy,\text{cal}} \) in the above expression is evaluated from,

\[
\sigma_{bcy,\text{cal}} = \frac{12M}{bt^3}
\]  

(6)

where \( M \) = bending moment in the plate element; \( t \) = thickness of the plate; \( b \) = width of the plate.

To avoid buckling of the flanges of the girder, stiffeners running longitudinally throughout the length of the bridge are provided. These stiffeners undergo bending as continuous beams due to the loading on the bridge deck. The design procedure is similar to the one used before.

The next step is the design of diaphragm walls. Diaphragms are designed as plate girders with flange width equal to 20 times the thickness of the deck flanges. Each diaphragm is subjected to a loading intensity equal to the product of the load per unit area on the bridge and the center-to-center distance of the diaphragms. Vertical stiffeners are designed to avoid buckling of the flanges.

**5. Analysis and Design of Cable-Net Bridge**

**5.1 Configuration**

With minor modifications, the basic configuration of the pylons and the deck girder are taken to be the same as that of the cable-stayed bridge. All the cables in the side-spans are connected to the top of the tower instead of distributing them over the length. This is done to counterbalance the load coming on the tower from the main span through the single suspension cable coming and joining at the top of the tower. The important parameters in decision-making regarding the cable-net configuration and their optimal values are: a) Dip of suspension cable = 90m.; b) Rise of prestressing cable = 28m; c) Diagonal ties are used for prestressing; d) Deck panel length = 20m; e) For the side-spans, inclined cables are used without any stiffening with center-to-center distance of 20m.

**5.2 Loading, Analysis and Design Methodology**

The loading, analysis and design methodology are similar to that of the cable-stayed bridge. The optimal pretension to be given is again determined using the Flexibility approach for
the vertical hangers. The corresponding pretension forces in the inclined ties of the cable-net are determined by an iterative approach.

6. Comparative Analysis

Figs. 3a and 3b show the deck bending moment, pylon axial force and pylon bending moment for the cable-stayed and cable-net bridge systems, the designs of which are given in Figs. 4 & 5 respectively. The weights of steel used along with the L/d ratios (unsupported length to deflection ratios) are presented in Table 1. The cable areas and the pretensioning forces are given in Tables 2 & 3. The following trends can be seen on examining the presented data:

1. Deflections in the main span of the cable-net bridge are generally less than half of those for the cable-stayed bridge. This clearly demonstrates the increased stiffness of the net as compared to individual cable configurations. This increased stiffness leads to reduced bending moments in the deck girder enabling the deck box-girder depth to be reduced to almost half of that of the cable-stayed bridge. This also leads to a corresponding reduction in dead load.
2. The absolute maximum values of the bending moments and axial compressive forces in the pylons are of the same order of magnitude in the two cases. However, while the pylon bending moments in the cable-stayed bridge are uniform right from the top necessitating the use of uniformly large cross-sections, those in the cable-net bridge increase gradually from the top with the maximum occurring close to the water-line, thus enabling the possible use of tapered cross-sections.
3. For the cable-stayed bridge, a large unsupported length is to be used in the centre of the span. This is because in this region, the cable areas increase exponentially and it would be uneconomical to provide cables right up to the centre of the bridge. It is this unsupported length which causes large bending moments in the deck and governs the design. This problem is effectively countered in the cable-net bridge by the use of vertical hangers.
4. The basic problem with cables, hindering their usage in engineering structures, has been their sagging and consequent loss of stiffness leading to the weakening of the structure. The high stiffness achieved by the use of a net provides an effective solution to this. Such cable-nets could thus be used not only for long-span bridges but also for other cable-supported structures.

7. Conclusions

An innovative Cable-Net bridge system using fundamental concepts from suspended roof structures has been proposed. Based on a comparative analysis of this system and of existing systems, the following conclusions can be made:
1. The increased stiffness of the cable-net system leads to smaller deflection and bending moments. The resulting reduction in weight of steel involved makes the proposed system viable too. Therefore, in the intermediate span range of 500m-1500m, the proposed system is superior to both the cable-stayed and the cable-suspended systems.
2. The algorithm proposed herein gives an efficient and simple way for evaluating the optimal cable pretension in both the new and existing systems.

8. References

Cable-Stayed Bridges. *International Conference of Cable-Stayed Bridges*, Bangkok.


Table 1: Comparison of Weight of Steel Used and Deflections of Cable-Stayed (C-S) Bridge and Cable-Net (C-N) Bridge

<table>
<thead>
<tr>
<th>Weight of Steel (kg)</th>
<th>C-S</th>
<th>C-N</th>
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<tbody>
<tr>
<td>1. Cables</td>
<td>1.04*10^6</td>
<td>1.945*10^6</td>
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<td>2. Pylons</td>
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<tr>
<td>Box-section</td>
<td>1.598*10^6</td>
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<td>stiffeners</td>
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<td>Total for each pylon</td>
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<td>3. Deck</td>
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<td>Box-section</td>
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<td>Longitudinal stiffeners</td>
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<td>Diaphragm walls</td>
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<td>Vertical stiffeners for diaphragms</td>
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<td>Vertical stiffeners for girder webs</td>
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<td>Total for deck girder</td>
<td>18.464*10^6</td>
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<td>Deflection (Ratio of unsupported length to maximum deflection, L/d)</td>
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Table 2: Nominal Diameter (mm) and Pretension (*10^6 N) of Cables in Cable-Stayed Bridge

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Table 3: Nominal Diameter (mm) and Pretension (*10^6 N) of Cables in Cable-Net Bridge

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Fig. 1a. Cable-Suspended Bridge System

Fig. 1b. Cable-Stayed Bridge System

Fig. 1c. Cable-Net Bridge System Proposed

1. Suspension Cable
2. Stay-Cable
3. Pylon
4. Vertical Hanger
5. Deck Girder
6. Inclined Tie
7. Pretension Cable

Fig. 2. Load Cases For Live Load
Fig. 3a. Force Diagrams for Cable-Stayed Bridge
Fig. 3b. Force Diagrams for Cable-Net Bridge
Acknowledgements

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