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POSTER SESSION 4

Innovative Bridge Structures

Structures nouvelles de ponts

Neuzeitliche Brückentragwerke

Coordinator: R.S. Stilwell, Canada

Bridge Strengthening by Post-Tensioning

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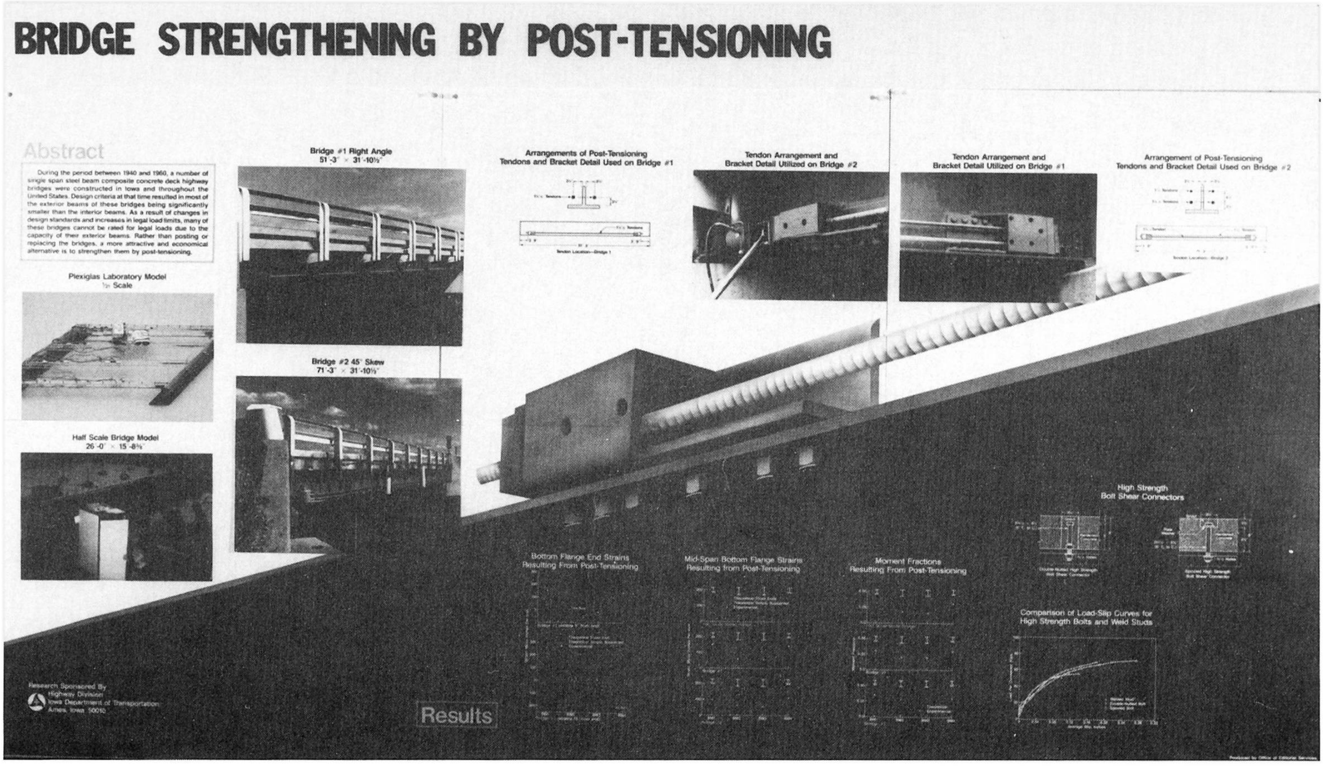
During the period between 1940 and 1960, a number of single span steel beam composite concrete deck highway bridges were constructed in Iowa and throughout the United States. Design criteria at that time resulted in most of the exterior beams of these bridges being significantly smaller than the interior beams. As a result of changes in design standards and increases in legal load limits, many of these bridges cannot be rated for legal loads due to the capacity of their exterior beams. Rather than posting or replacing the bridges, a more attractive and economical alternative is to strengthen them. With research grants from the Iowa Department of Transportation and the Iowa Highway Research Board, a method of strengthening these bridges by post-tensioning has been developed. This study was divided into three phases. Phase I involved the development of bracket designs and the post-tension strengthening and testing of a half-scale model bridge in the laboratory. Phase II - the majority of which was completed during the summer of 1982 - involved the strengthening and testing (before and after strengthening) of two actual bridges. The final phase of the study (Phase III) involves the periodic inspection of the two bridges, their retesting during the summer of 1984, and the development of a practical design methodology for the strengthening technique. The major finding of Phases I and II was that post-tensioning is an economical and viable strengthening method. As may be noted in the graphs, there was considerable end restraint in both bridges (Bridge No. 1 and Bridge No. 2). Thus the post-tensioning was more effective in the laboratory models than on the actual bridges (due to the fact that there was no end-restraint). The end restraint on Bridges No. 1 and 2, however, also reduced the live load stresses. Thus in effect the end restraint effect on live load stresses and post-tensioning stresses compensate each other. Phase III of the study has been completed and the final report is presently being prepared. Both bridges previously strengthened were retested. Very little change was noted in their behavior from that noted during their initial strengthening two years earlier. The retesting made possible the determination of loss of prestress in both bridges; Bridges No. 1 and No. 2 had losses of approximately 5% and 7% respectively. This loss is primarily due to relaxation of the end restraint previously noted. Through the use of orthotropic plate theory and finite element models, a practical design methodology for determining the required post-tensioning forces has been developed for the design engineer and is included in the final report being prepared.



BRIDGE STRENGTHENING BY POST-TENSIONING
Abstract
During the period between 1940 and 1960, a number of single span steel beam composite concrete deck highway bridges were constructed in Iowa and throughout the United States. Design criteria at that time resulted in most of the exterior beams of these bridges being significantly smaller than the interior beams. As a result of changes in design standards and increases in legal load limits, many of these bridges cannot be rated for legal loads due to the capacity of their exterior beams. Rather than posting or replacing the bridges, a more attractive and economical alternative is to strengthen them by post-tensioning.
Plexiglas Laboratory Model
Scale
Half Scale Bridge Model
26'-0" × 15'-8⅝"
Bridge #1 Right Angle
51'-3" × 31'-10½"
Bridge #2 45° Skew
71'-3" × 31'-10½"
Arrangements of Post-Tensioning
Tendons and Bracket Detail Used on Bridge #1
Tendon Arrangement and
Bracket Detail Utilized on Bridge #2
Tendon Arrangement and
Bracket Detail Utilized on Bridge #1
Arrangement of Post-Tensioning
Tendons and Bracket Detail Used on Bridge #2
High Strength
Bolt Shear Connectors
Bottom Flange End Strains
Resulting From Post-Tensioning
Mid-Span Bottom Flange Strains
Resulting from Post-Tensioning
Moment Fractions
Resulting From Post-Tensioning
Comparison of Load-Slip Curves for
High Strength Bolts and Weld Studs
Results
Research Sponsored By
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Transverse Prestressing of Prestressed Laminated Wood Bridge Decks

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A form of construction which is well suited to short span bridges is the nailed-laminated wood deck. However, experience has shown the load distribution of this type of deck is severely impaired with time due to rusting and ultimate failure of the nails. A method (1) has been developed for the rehabilitation of nailed laminated wood decks which involves introduction of transverse prestressing. This method has also been successfully applied to new construction. Initially, no standard method of design existed for such an application of prestress to wood systems and studies have been undertaken (1) to assess the behaviour of such systems and to determine the appropriate values of structural parameters for use in analysis and design. As shown in the figure, the details of the prestressing system vary depending on whether it is applied to rehabilitation of existing structures or to new construction. Typical details are provided in the Ontario Highway Bridge Design Code (2) which includes a section on prestressed wood systems.

Laboratory and field studies have been conducted on the system (1). The objective of the laboratory studies was the determination of orthotropc plate parameters and prestress losses. The main variables were type of wood (hem-fir, white pine and red pine), type of wood treatment and level of initial prestress. Tests were conducted on laminated beams and plates and on axially loaded prisms formed from laminates. The results of these investigations have also been reflected in the provisions of Reference (2).

The prestressed laminated wood deck lends itself to prefabrication. Its superior load distribution over that of existing conventional nailed laminated decks has been demonstrated (1) as shown in the figure. The Hebert Creek Bridge which was rehabilitated in 1976, has been monitored regularly, and has confirmed that the system is feasible and economical. This has also been confirmed at other sites in Ontario.

REFERENCES

1. Taylor, R.J., Batchelor, B.deV. and Van Dalen, K., "Prestressed Wood Bridges", Procs. International Conference on Short and Medium Span Bridges", Vol. 2, August 8-12, Toronto, Ontario, pp: 203-218.

2. Ontario Highway Bridge Design Code, Ontario Ministry of Transportation and Communications, Downsview, Ontario, 1983.

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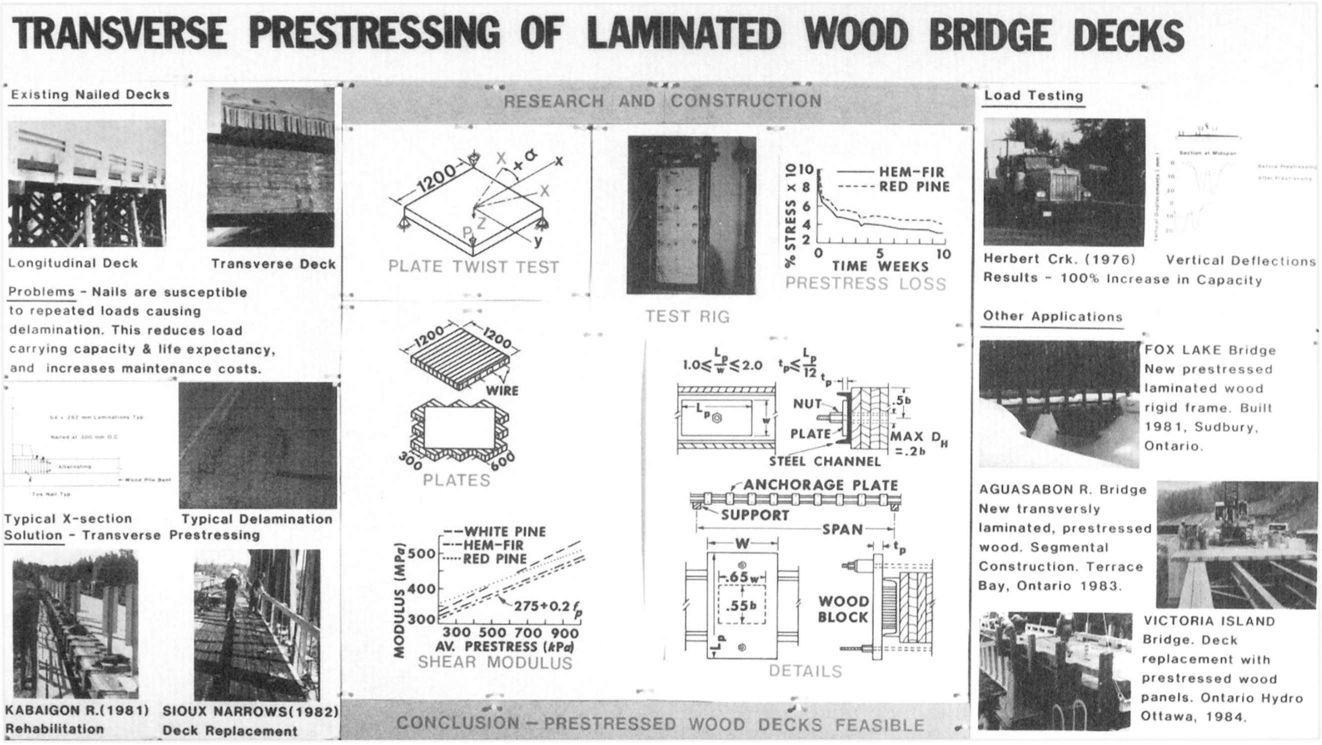
TRANSVERSE PRESTRESSING OF LAMINATED WOOD BRIDGE DECKS
Existing Nailed Decks
Longitudinal Deck
Transverse Deck
Problems - Nails are susceptible to repeated loads causing delamination. This reduces load carrying capacity & life expectancy, and increases maintenance costs.
Typical X-section
Typical Delamination
Solution - Transverse Prestressing
KABAIGON R.(1981) Rehabilitation
SIOUX NARROWS(1982) Deck Replacement
RESEARCH AND CONSTRUCTION
PLATE TWIST TEST
TEST RIG
HEM-FIR
RED PINE
% STRESS x 10
TIME WEEKS
PRESTRESS LOSS
WIRE
PLATES
WHITE PINE
HEM-FIR
RED PINE
MODULUS (MPa)
275+0.2 f_p
AV. PRESTRESS (kPa)
SHEAR MODULUS
$1.0 \leq \frac{L_p}{w} \leq 2.0$
$t_p \leq \frac{L_p}{12}$
NUT
PLATE
MAX D_H =.2b
STEEL CHANNEL
ANCHORAGE PLATE
SUPPORT
SPAN
WOOD BLOCK
DETAILS
CONCLUSION — PRESTRESSED WOOD DECKS FEASIBLE
Load Testing
Herbert Crk. (1976)
Vertical Deflections
Results - 100% Increase in Capacity
Other Applications
FOX LAKE Bridge New prestressed laminated wood rigid frame. Built 1981, Sudbury, Ontario.
AGUASABON R. Bridge New transversly laminated, prestressed wood. Segmental Construction. Terrace Bay, Ontario 1983.
VICTORIA ISLAND Bridge. Deck replacement with prestressed wood panels. Ontario Hydro Ottawa, 1984.

Tests and Analyses on the Pedestrian Suspended-Slab Bridge

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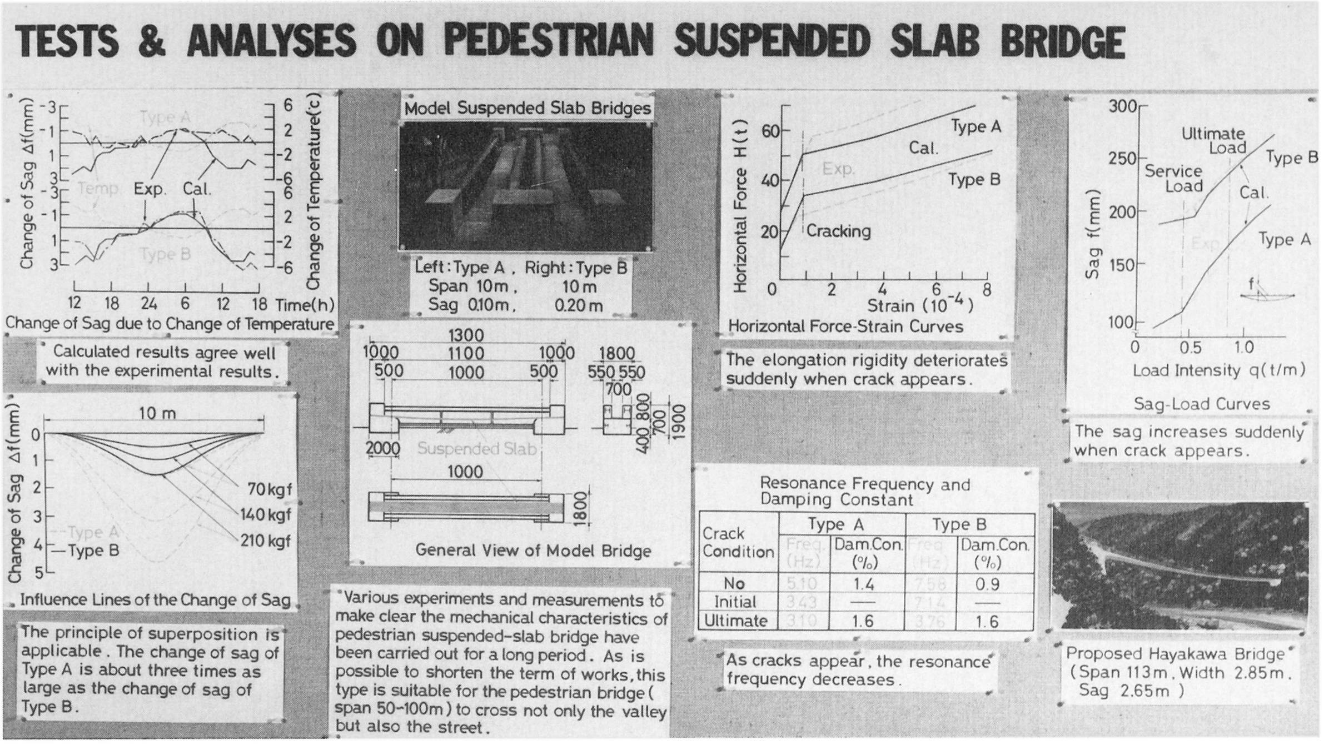
A suspended-slab bridge (Spannband-Brücke) is essentially the same structural system as the suspended-roof in buildings. This type bridge is made by spanning of tendons which are lined with reinforced concrete to provide the rigidity as slab. Its advantage is not only applicable to long span, but also unnecessary to use the elements such as main towers, hangers and stiffened members in a conventional suspension bridge, and almost free from the maintenance works. However, these bridges are very few, because we have only an insufficient knowledge on their characteristics of deflection or vibration and the effects of cracking about such a structure.

For these reasons, authors have conducted following experiments; firstly, to construct two model bridges of this type for pedestrians with a span of 10m and a width of 0.7m which were designed by using the span/sag ratio, 100 (Type A) and 50 (Type B), respectively; secondly, to investigate the behaviour of these bridges due to temperature variations and pedestrian-actions; thirdly, to measure the cracks relating to the sand-loading test; and finally, to execute the shaking test.

The main points obtained from the experimental results are summarized as follows; 1) cracks appeared in the slab near the bridge seat due to the mere change of temperature; 2) when static loading up to about 1.0t/m, which is 4.1 times as large as the design load, were applied, the number of cracks appeared in the slab was 77 in Type A per span length 10m (mean crack spacing ℓ_{mean}=13cm; maximum crack width w_{max}=0.8mm appeared near the bridge seat); and 71 in Type B (ℓ_{mean}=14cm, w_{max}=0.7mm), respectively, but all these cracks closed after removing the load; 3) although the vibration mode was close to bending vibration in the case of no crack or few cracks, the mode approached to longitudinal vibration of tendons as the number of cracks were increasing, and the resonance frequency had a tendency to decrease; 4) smaller span/sag ratio is not only favorable for all mechanical properties such as deflection, vibration, cracking et al., but also economical.


TESTS & ANALYSES ON PEDESTRIAN SUSPENDED SLAB BRIDGE
Change of Sag Δf(mm)
Change of Temperature(°c)
Type A
Temp.
Exp.
Cal.
Type B
12 18 24 6 12 18 Time(h)
Change of Sag due to Change of Temperature
Calculated results agree well with the experimental results.
10 m
Change of Sag Δf(mm)
70kgf
140kgf
210kgf
Type A
Type B
Influence Lines of the Change of Sag
The principle of superposition is applicable. The change of sag of Type A is about three times as large as the change of sag of Type B.
Model Suspended Slab Bridges
Left: Type A, Right: Type B
Span 10m, 10m
Sag 0.10m, 0.20m
1300
1000 1100 1000 1800
500 1000 500 550 550
700
2000
Suspended Slab
1000
400 800 700 1900
1800
General View of Model Bridge
Various experiments and measurements to make clear the mechanical characteristics of pedestrian suspended-slab bridge have been carried out for a long period. As is possible to shorten the term of works, this type is suitable for the pedestrian bridge (span 50-100m) to cross not only the valley but also the street.
Horizontal Force H(t)
Type A
Cal.
Exp.
Type B
Cracking
Strain (10^{-4})
Horizontal Force-Strain Curves
The elongation rigidity deteriorates suddenly when crack appears.
Resonance Frequency and Damping Constant
Crack Condition | Type A Freq. (Hz) | Type A Dam.Con. (%) | Type B Freq. (Hz) | Type B Dam.Con. (%)
No | 5.10 | 1.4 | 7.58 | 0.9
Initial | 3.43 | — | 7.14 | —
Ultimate | 3.10 | 1.6 | 3.76 | 1.6
As cracks appear, the resonance frequency decreases.
Sag f(mm)
Ultimate Load
Service Load
Type B
Cal.
Exp.
Type A
Load Intensity q(t/m)
Sag-Load Curves
The sag increases suddenly when crack appears.
Proposed Hayakawa Bridge (Span 113m, Width 2.85m, Sag 2.65m)

Cable-Stayed Bridge with New Vierendeel Type Girder

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An example of a two-dimensional section model in wind-tunnel.
The results of the experiments of spring-mounted section models on torsional oscillation.
Spring-mounted section models were stable against torsional oscillation. Flutter or vortex excited oscillation did not occur until the wind speed reached 110 m/s. This figure shows V-δ (wind speed --- logarithmic decrement) curves at the torsional double amplitude of 2°. For reference, V-δ curves for ordinary truss-type girders under the same conditions are also indicated.

The maximum and minimum bending moments of a Vierendeel-type girder against the changes in parameter K when a uniform load of 1.0 t/m is applied to the full span. The cross-sectional area of the girder A = 0.240 m^2, the moment of inertia I = 8.5054 m^4; A_t of the main tower = 0.780 m^2; I_t of the main tower = 7.250 m^4; Ac of cable = 0.01893 m^2 (average); and the overall length of the girder L = 884 m.

$$K = EI/L^2 EcAc$$

The characteristics of the changes in maximum and mimimum bending moments due to the changes in the flexural rigidity of the girder K, are illustrated. When K is smaller than $10^{-3} \sim 10^{-4}$, the changes in the bending moment are not so remarkable.

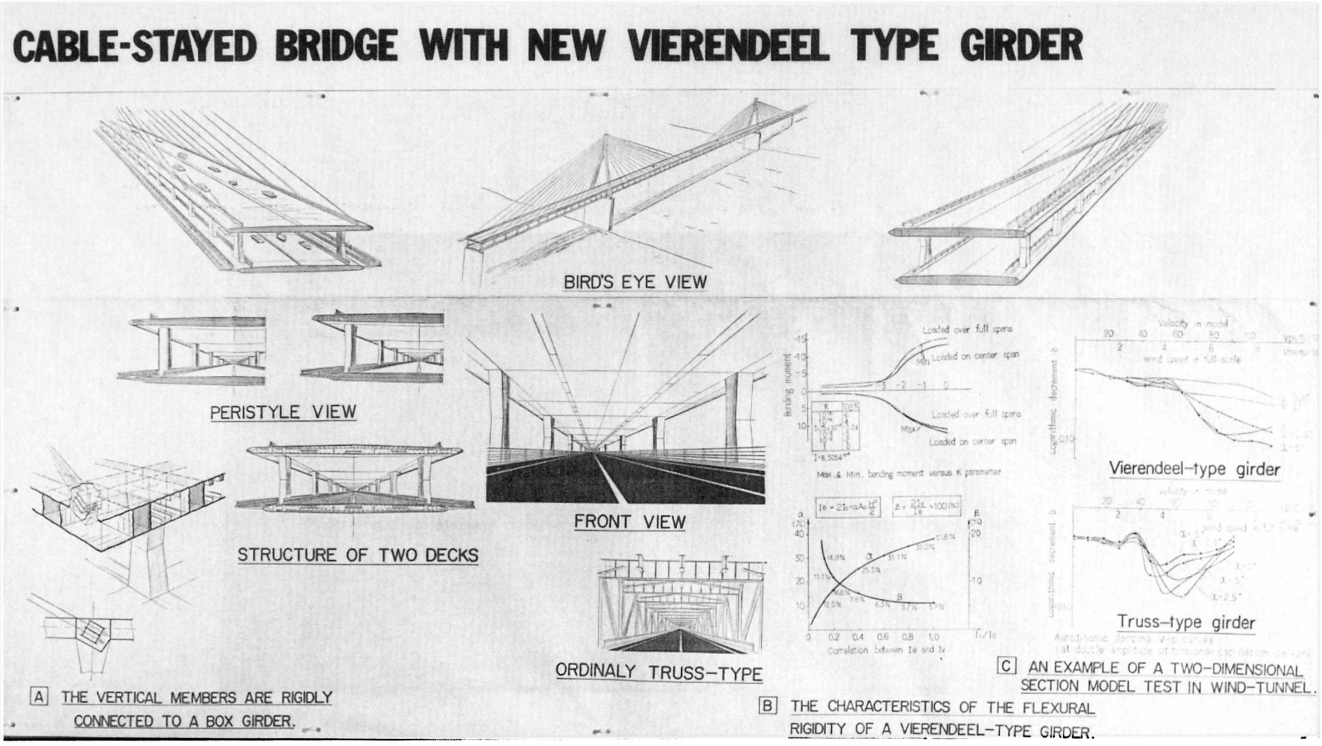
The characteristics of the flexural rigidity of a Vierendeel-type girder observed when the flexural rigidity of the vertical members of the girder is changed. When parameters α and β are used, the equivalent moment of inertia of the girder Ie is expressed by

$$Ie = 2Io + \alpha Ao \left(\frac{H}{2} \right)^2$$

$$\beta = \frac{2Io}{Ie} \times 100\ (\%)$$

Where, Io is the moment of inertia of upper and lower chord members, Ao is the cross sectional area of upper and lower chord members, H is the distance on centers of upper and lower chord members, and α is the equivalent section coefficient.

The design concept when the vertical members of a Vierendeel-type girder are connected to a box girder.
The vertical members receive both the bending moment action as the stiffening girder in the plane of the girder and the deformation resisting action of the cross section of the whole stiffening girder in the plane at transverse direction to the plane of the girder. Therefore, the connection between a vertical member and a box girder must be rigid and secure.


CABLE-STAYED BRIDGE WITH NEW VIERENDEEL TYPE GIRDER
BIRD'S EYE VIEW
PERISTYLE VIEW
STRUCTURE OF TWO DECKS
FRONT VIEW
ORDINALY TRUSS–TYPE
A THE VERTICAL MEMBERS ARE RIGIDLY CONNECTED TO A BOX GIRDER.
B THE CHARACTERISTICS OF THE FLEXURAL RIGIDITY OF A VIERENDEEL–TYPE GIRDER.
C AN EXAMPLE OF A TWO–DIMENSIONAL SECTION MODEL TEST IN WIND–TUNNEL.
Vierendeel–type girder
Truss–type girder
Loaded over full spans
Loaded on center span
Bending moment
Velocity in model
Correlation between Ie and Iv

Innovative Design for Main Towers of Long Span Suspension Bridges

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Erection of main shaft

Generally speaking, when larger unit blocks are used to reduce the construction period of a main shaft or to speed up the work, not only is the unit weight increased but also larger manufacturing, transportation and erection equipment or facilities are required. The larger erection cranes (creeper cranes) which become necessary, in particular, will prove to be inconvenient.

It is proposed that the construction period be reduced by assigning many general-purpose cranes of relatively small capacity to each main shaft (for example, four 30t jib cranes for each main shaft).

As sufficient flexibility can be obtained by selecting an appropriate rigidity ratio for columns and beams even if a slender shaft is used, a wider space which makes the work easier is produced.

Top saddle

The vertical loads of main cables concentrated on the center of the saddle must be distributed on the columns around a main shaft. If a saddle beam of which the supporting point is on the periphery of a main shaft is used instead of a saddle and several cross beams made of steel plates are installed at right angles to the saddle beam to support it, the reaction forces of the saddle beam made of steel plates are installed at right angles to the saddle beam to support it, the reaction forces of the saddle beam are transferred to the periphery of the main shaft. In this case, it is necessary to distribute uniformly the reaction forces of the saddle beam on the periphery of the main shaft through the cross beam. This can be made possible by changing the size of the cambers which are fitted to the cross beams --- larger cambers near the periphery of the main shaft and smaller ones near the center of the cross beam.

Observation tower

As the inside of the main shaft is very large, it may be used as an observation tower, leisure facility, or for some other purpose.

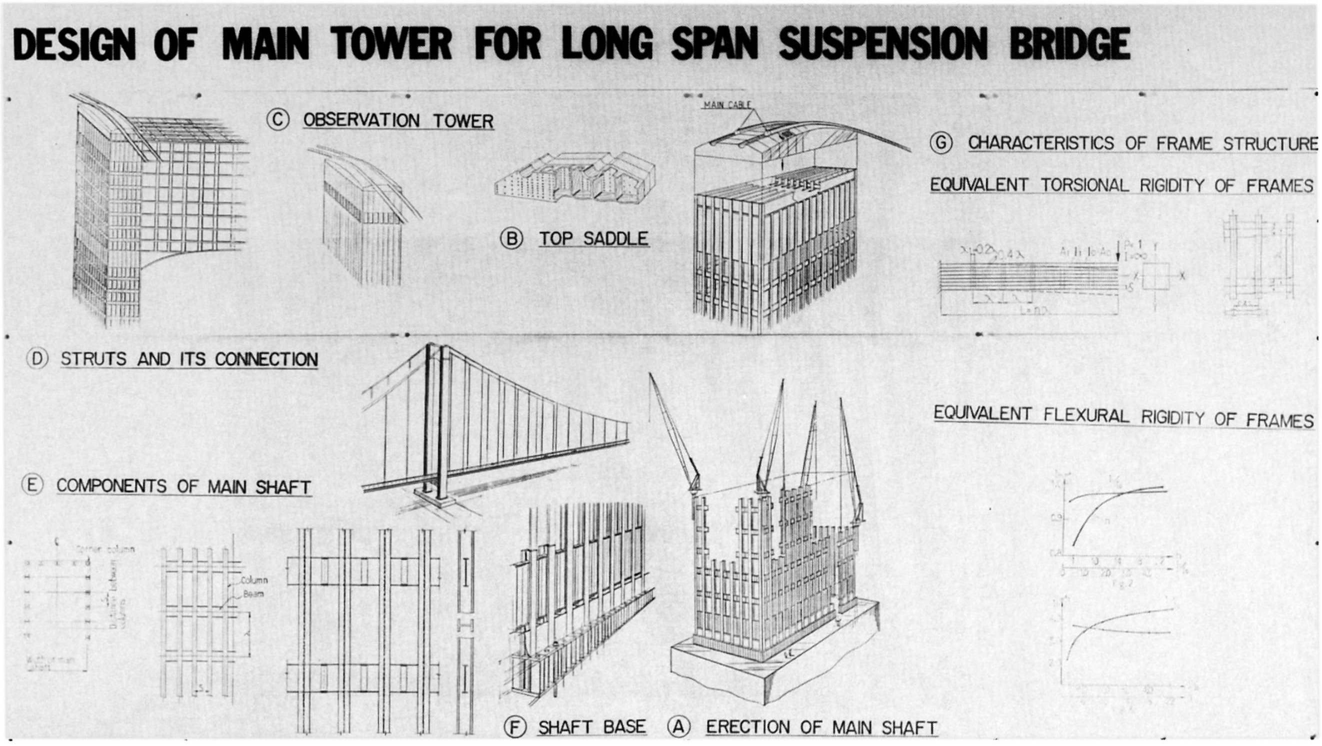
Struts and its connection

Although the width of a strut is necessarily large to match the width of the main shaft, struts should be of a rigid frame structure. When members are connected, the transmission of each members stress must be made certain.

Components of main shaft

The main members constituting a main shaft are called columns or column members. Since heavy H-shaped steel sections for columns are used as main members.

As the vertical loads of main cables working on the top saddles of a long span suspension bridge are very large, columns with a large cross-section are necessary. Some catalogue size typical heavy H-shaped steel sections for columns are used.


DESIGN OF MAIN TOWER FOR LONG SPAN SUSPENSION BRIDGE
C OBSERVATION TOWER
B TOP SADDLE
MAIN CABLE
G CHARACTERISTICS OF FRAME STRUCTURE
EQUIVALENT TORSIONAL RIGIDITY OF FRAMES
D STRUTS AND ITS CONNECTION
EQUIVALENT FLEXURAL RIGIDITY OF FRAMES
E COMPONENTS OF MAIN SHAFT
Column
Beam
F SHAFT BASE
A ERECTION OF MAIN SHAFT

Connaught Bridge Replacement, Vancouver, BC

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The Connaught Bridge in Vancouver, Canada crosses a navigable tidal inlet which is approximately 165m wide. Other constraints such as property ownership, requirements of navigation and road and rail overpass clearances dictated the final bridge length of 1100m including a main span of 84 metres. Typical spans are between 33 and 39 metres in length.

Because the structure crosses a major urban redevelopment area which will also be the site for Exposition 86, i.e. a major "people" place, the clients were willing to pay a premium for an aesthetically pleasing structure. The chosen design utilizes a cast-in-place spine beam supporting precast, pretensioned "wing" elements which act as permanent cantilever forms for infill deck concrete. A typical cross-section is shown in Figure 1. After the infill concrete is placed, the structure is post-tensioned longitudinally using a combination of 12 and 19 -15mm strand tendons. The composite deck is post-tensioned transversely to carry the applied live loads. The superstructure is continuous between expansion joints which are spaced between 250 and 300 metres apart.

Ground conditions at the site are generally poor with loose fills overlying soft clays and silts to a depth of 10 metres. Expanded base piles formed on glacial drift were used to support the structure. In a departure from conventional expanded base piles, a permanent steel liner was placed and filled with normal concrete to give better lateral resistance to seismic forces.

The cost of the structure based on tenders received is about C$950 per m2.

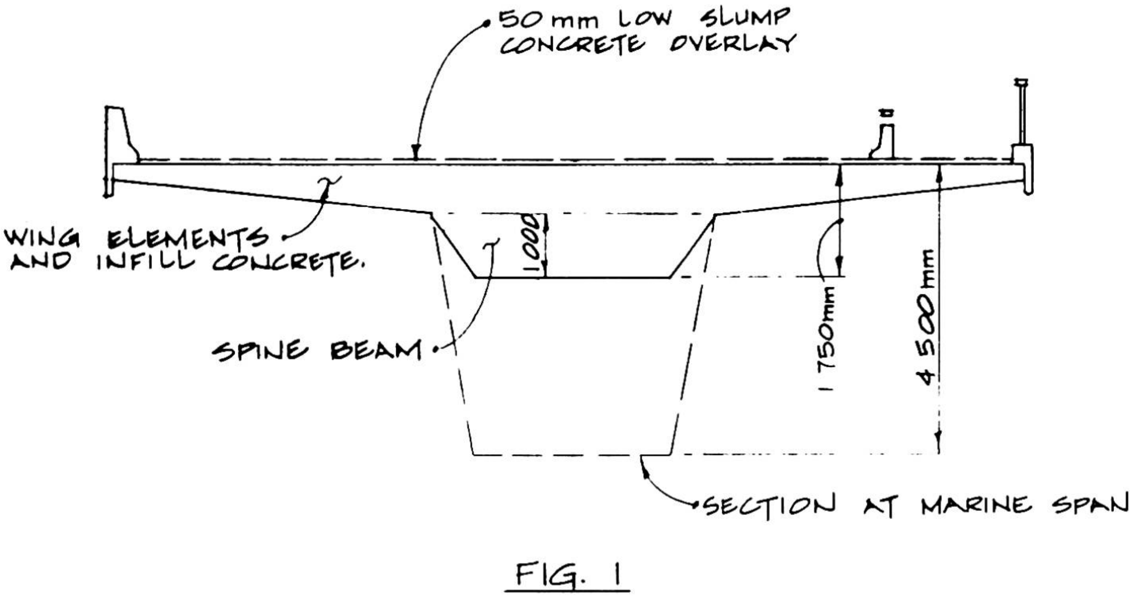
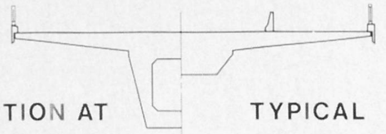
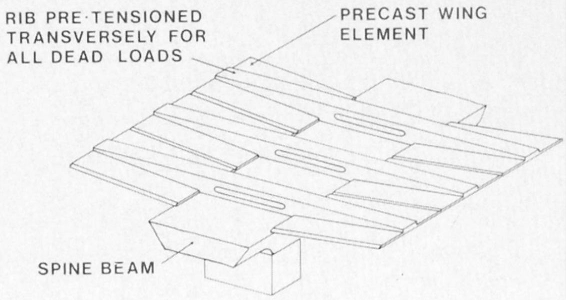


FIG. 1

CONNAUGHT BRIDGE REPLACEMENT, VANCOUVER, B.C.



WING ELEMENTS



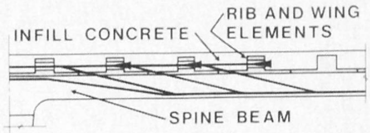
SPINE BEAM AND WING ELEMENTS

COMPLETED STRUCTURE

CONNAUGHT BRIDGE

OWNER - CITY OF VANCOUVER
COST - $ 35,000,0 00

The cast-in-place spine beam supports precast, pretensioned 'wing' elements which act as permanent cantilever forms for infill deck concrete. After the infill concrete is placed, the structure is post-tensioned longitudinally, using a combination of 12 and 19 - 15mm dia. tendons. The composite deck is post-tensioned transversely for applied live loads.



SECTION
LONGITUDINAL POST-TENSIONING

Aerodynamic Stability of Twin Suspension Bridge Concept

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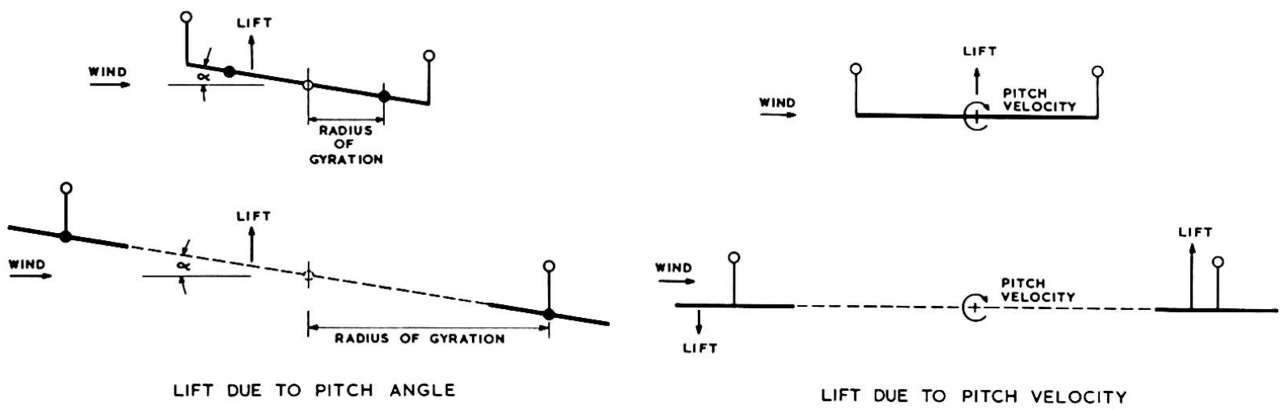
THEORY

When a conventional road deck is twisted the wind causes a negative aerodynamic moment, which reduces the torsional stiffness and frequency as the wind speed increases. At some critical wind speed this frequency coincides with that of bending, and the two modes couple together in an unstable oscillation called "flutter". This speed can be raised by increasing the torsional stiffness with a lattice-truss or steel box. Unfortunately this increases the dead load carried by the cables, and becomes uneconomic for very long spans.

If the still-air torsion frequency could be reduced to that of bending the two frequencies would never coincide in high winds, and flutter would be avoided. However, high torsional stiffness would still be needed because the torsion frequency would reduce to zero at some wind speed, leading to another instability called "static divergence". Such a solution could therefore be achieved only by increasing the torsional inertia of the deck. Even if this was practically possible, the torsional damping would be nearly zero because the aerodynamic lift due to pitch velocity acts at the centre of the deck, and so gives no damping moment.



Torsional stiffness can be achieved, without weight penalty, by spacing the cables much further apart. If at the same time, the two deck halves are also separated to hang directly under each cable leaving a huge "slot" between them, three phenomena occur. With the understanding that the two decks are constrained to move as a single body by rigid transverse beams at intervals along the span, then

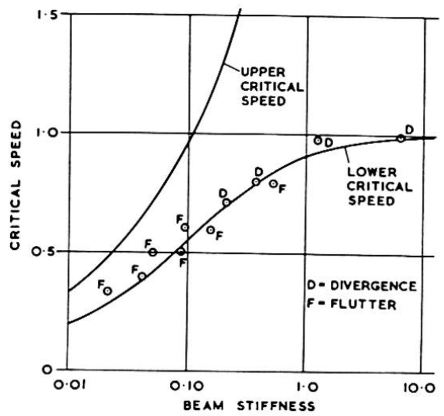
i) the destabilising aerodynamic moment remains exactly the same, so that no increase in torsional stiffness is needed,
ii) the still-air torsional frequency is reduced to that of bending by the increased radius of gyration
and iii) the aerodynamic damping in the torsion mode becomes highly positive.

Such a bridge will not "flutter" at any speed, and has high aerodynamic damping in both bending and torsion modes. It can only become unstable in a static divergence mode, whose critical wind speed can be raised to any value by increasing the distance between the decks.

EXPERIMENT

The theoretical predictions of the aerodynamic forces on twin decks have been confirmed by tests on wind-tunnel models. Further tests on an aeroelastic section model with a wide range of transverse beam stiffnesses were then conducted. Very rigid beams gave the predicted divergence speed with high subcritical damping and no flutter. More flexible beams reduced the divergence speed, and very flexible beams led to independent flutter of the two decks as would be expected. These results are shown below.

The weight of beams sufficiently stiff to avoid significant reduction of the critical divergence speed can, however, be shown to be only a few percent of that of the superstructure.



REDUCTION OF DIVERGENCE SPEED DUE TO
TRANSVERSE BEAM FLEXIBILITY
AND COMPARISON WITH EXPERIMENT

PRACTICE

Various practical forms of twin bridge can be envisaged. A pair of supporting cables for each deck is the most likely configuration, with the transverse beams attached directly below the deck or connecting the four cables above it. Precise equality of the still-air bending and torsion frequencies has been shown by experiment to be unnecessary.

TRANSVERSE BEAM

P. TVEIT

Dr. Eng., Aalborg Univ. Centre, Aalborg, Denmark

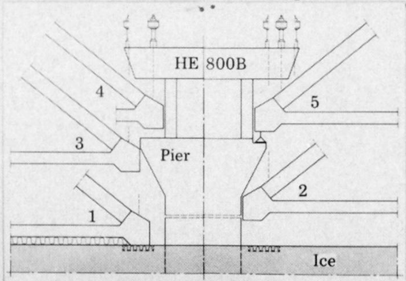
BUILDING NETWORK ARCHES ON REINFORCED ICE BETWEEN PIERS

Bridge for rivers and lakes in arctic areas

This poster deals with an investigation into the possibilities of using a reinforced layer of ice for casting the tie and erecting the structural steel for a network arch. When the longitudinal cables have been stressed, the span can be lifted from the ice up to its final position.

To facilitate formwork and insulation, the under side of the tie has been made flat. To facilitate removal of snow from the finished bridge, longitudinal edge beams have been almost removed. To reduce costs, each arch is a single universal column. If built, the proposed network arch would be more slender than any other arch bridge.

For the proposed span four sets of hangers are warranted to reduce bending in the chords. With the usual stiffness of arches, two sets of hangers will normally give sufficient reduction of bending.



Stages in lifting of span from ice to pier

Bridges in cold regions

Due to sparse habitation arctic roads usually carry little traffic. Still bridges for these roads must be able to carry heavy loads.

The present span has been designed for Danish loads and codes. The heaviest vehicle has 3 axles 1.5m apart and weigh 780 kN. It has been assumed that heavy loads do not occur often enough to cause fatigue.

In arctic regions it is often difficult and costly to keep the building workers employed in winter, so there should be an interest in a structure that leads to more winter work without loss of economy.

When advocating the network arch the author has heard that 50% of savings in steel weight is more than offset by high cost of fabrication and erection. The author points out that 82% of the structural steel in this proposed span is standard profiles. Their fabrication is not costly, but their use contribute to a slightly higher steel weight.

In climates where sufficiently thick ice covers cannot be relied upon, the arch and hangers, supplemented by a light temporary lower chord can be erected on 0.8m thick ice before the span is lifted on to the piers. In the spring the concrete lane can be cast on the temporary lower chord.

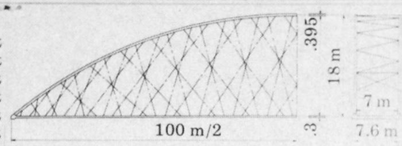
In warmer climates the finished span weighing 540t, can be lifted by pontoons. Then the lifting forces can be applied up to 1m from the the ends of the span.

Complete calculations and drawings can be ordered from the author.

Materials needed:

Universal columns	55 t
Windbracing HE 140A	5 t
Steel ropes	6 t
Ends of hangers	4 t
Other structural steel	3 t
Prestressing steel	9 t
Reinforcing steel	46 t

$$\frac{\Sigma\ 128\ t}{7 \times 100} = .183\ t/m^2 \qquad \text{Slenderness}\ \frac{100}{.395 + .3} = 144$$



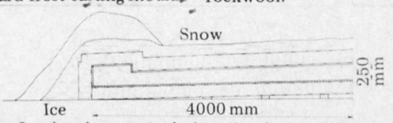
Building on reinforced ice

The ice cover of a lake or river can be reinforced by wood or ribbed bars put on top of the ice early in the winter.

Then the ice can be made 2.5m thick in stages by pumping water on to the ice, or faster by spraying water in the cold air above the ice.

Methods for casting the tie will depend on local climate and usage. In hard frost casting should take place in a well insulated tent that slides along the ice.

If the rockwool insulated tie is covered by snow, the temperature between snow and insulation will be 0°C regardless of ambient temperature. Thus it is not difficult to keep the tie warm once it is cast and covered. Curing heat will keep the deck warm for about 60 days if it is surrounded by .2m of rockwool.



Insulated concrete deck on ice during curing

B. BAKHT — T. THARMABALA
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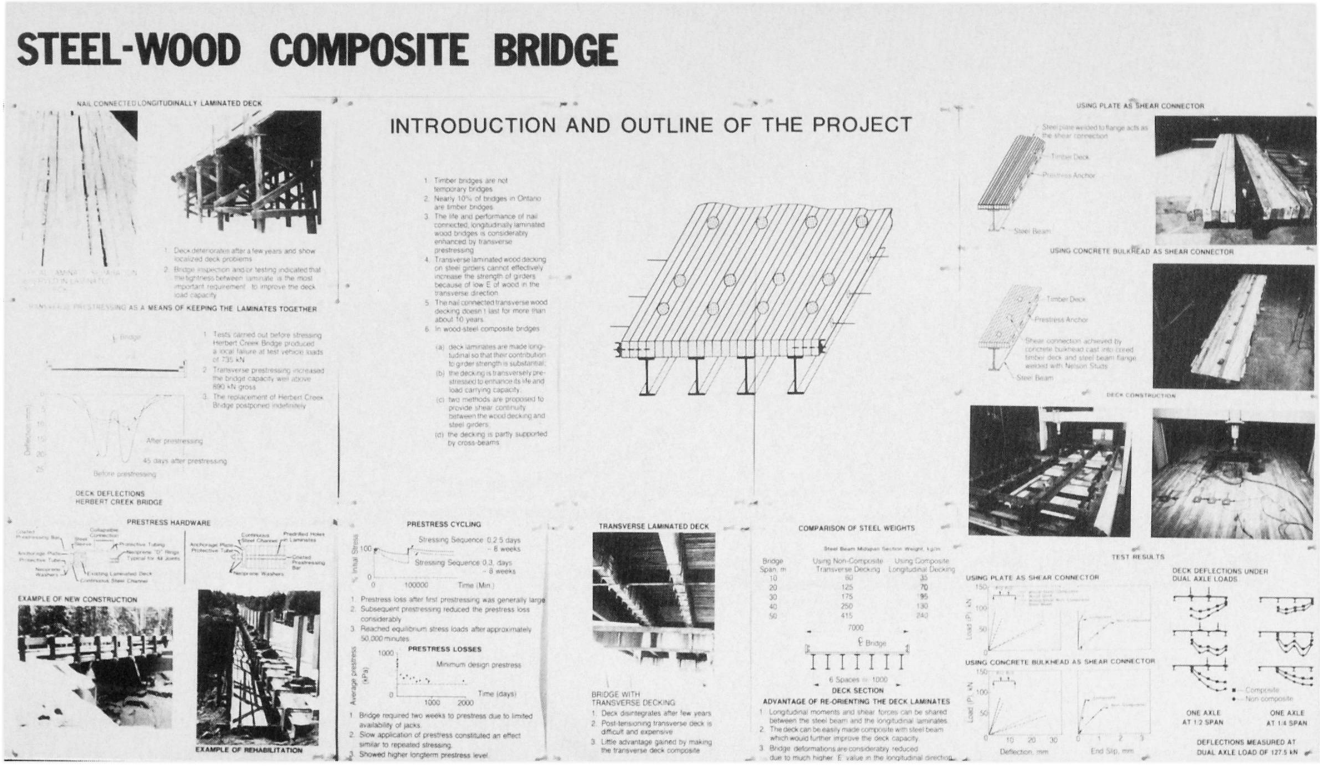

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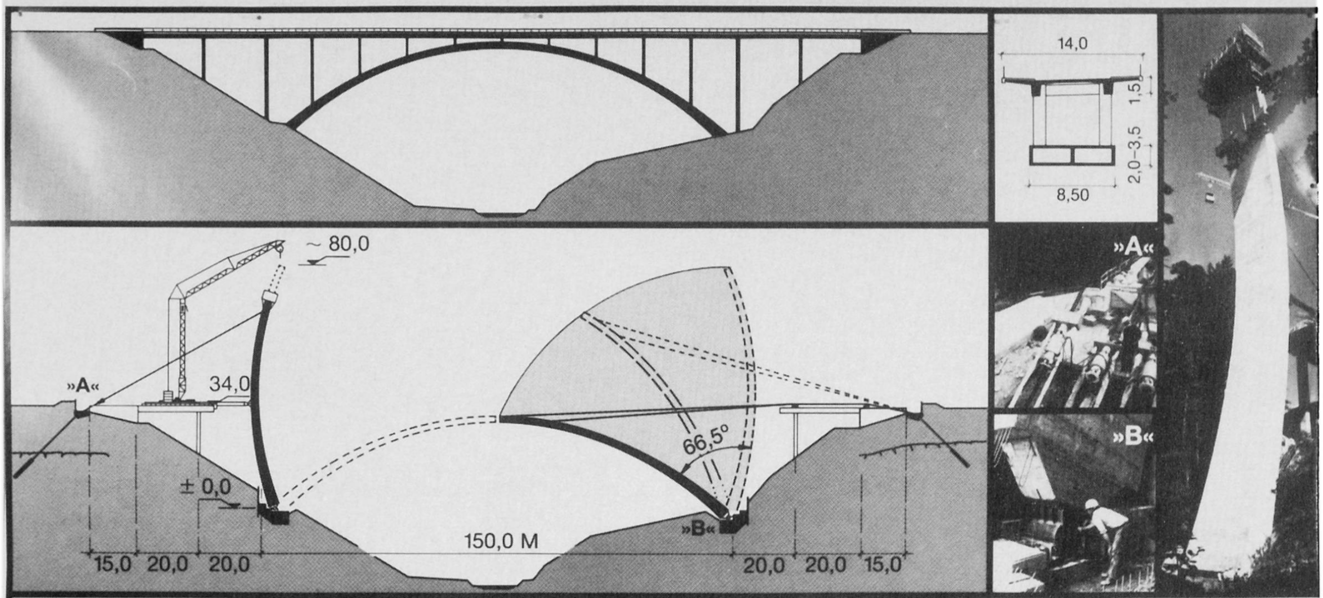
ARGENTOBEL BRIDGE, F.R.G.: NEW CONSTRUCTION METHOD FOR ARCHES

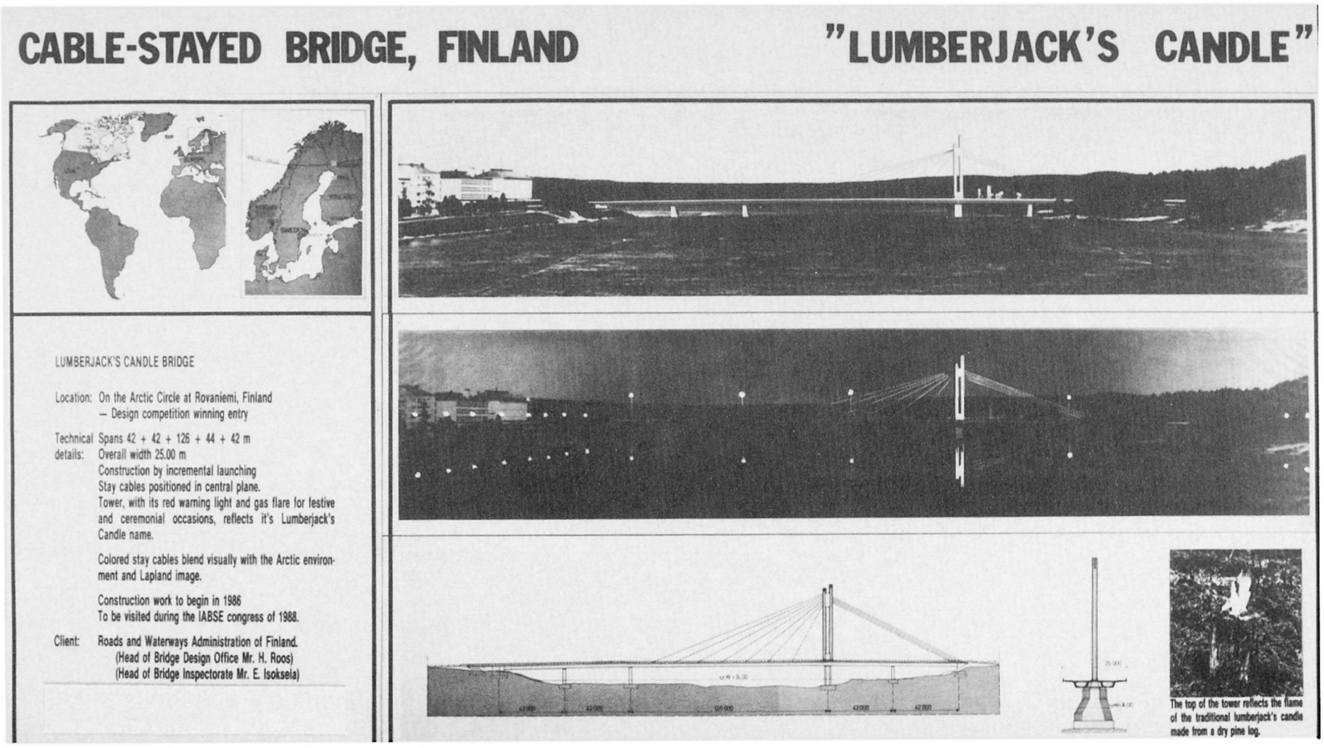


CONCRETING: accuracy ±3 cm, climbing formwork, 27 sections 81,5 m within 3 months

LOWERING: time required 5 days, angle 66,5° articulated steel bearing with teflon sliding surface, lowering cables consisting of strand tendons St 1570/1770

Locking: aligning operations by jacks at the crown and the arch springing, concreting of the end sections, removal of the articulated bearings an jacks

A. KETTUNEN — E. JARVENPAA
Suunnittelukortes Ltd OY, Oulu, Finland



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