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POSTER SESSION 1

Structural Design Process

Processus du project

Der Entwurfsprozess

Coordinator: R.S. Stilwell, Canada

Distribution of Wheel Loads on Highway Bridges

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The current criteria for the distribution of wheel loads in the U.S. bridge specifications have been undergoing change and expansion for over 50 years. The changes have primarily been introduced as modifications for a specific bridge type or condition with variations in the factors considered. As a result, the approach to the criteria has varied and resulted in inconsistencies in the codes. There now is a need for a complete review of load distribution in bridges recognizing a consistent approach to all bridge types and the availability of high speed computation.

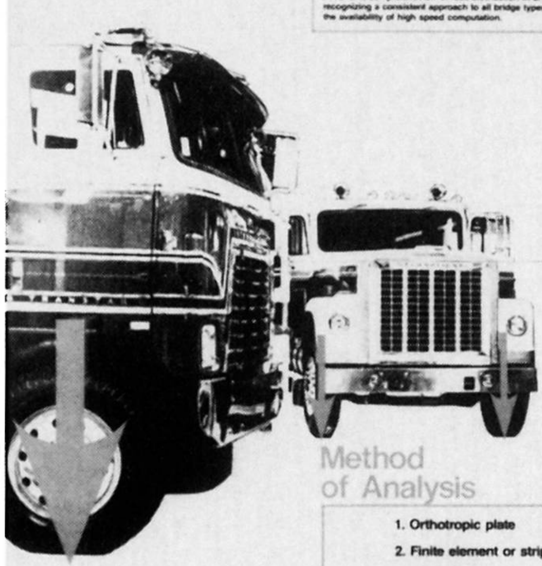
There are a number of methods of analysis that can be used to develop load distribution behavior. These methods include: orthotropic plate, finite element or strip, grillage analogy, folded plate, influence surfaces. Using the selected methods, the effects of aspect ratio, bridge stiffness parameter, edge effects, load position, skew, continuity and diaphragms need to be evaluated for the broad types of bridges.

This study is needed and should result in a consistent criteria format based on similar parameters. It should consider all factors which affect behavior. The option should be available and encouraged to use one of the theories for complex structures, while providing a simple format for simple bridges.

DISTRIBUTION OF WHEEL LOADS ON HIGHWAY BRIDGES

Abstract

The current criteria for the distribution of wheel loads in the U.S. bridge specifications have been undergoing change and expansion for over 50 years. The changes have been primarily introduced as modifications for a specific bridge type or condition with variations in the factors considered. As a result, the approach to the criteria has varied and resulted in inconsistencies in the codes. There now is a need for a complete review of load distribution in bridges recognizing a consistent approach to all bridge types and the availability of high speed computation.



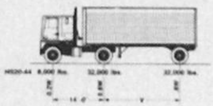
Design Criteria

STANDARD SPECIFICATIONS
for
HIGHWAY BRIDGES

The American Association of State Highway and Transportation Officials

DIVISION I — DESIGN

1983 DRAFT COPY



Percentage of Live Loads:

One or two lanes	100%
Three lanes	90%
Four lanes or more	75%

Traffic Lanes:
12 ft. wide lanes (with 10 ft. wide trucks), spaced across the entire bridge roadway width. Lanes shall be placed in numbers and position to maximize effect.

Interior Beams: Wheel Load Fraction (typical)

Kind of Floor	Bridge Designed for One Traffic Lane	Bridge Designed for Two or More Traffic Lanes
Timber: 6" Glued Laminated Panels on Glued Lam. Stringers	S/6.0	S/5.0
Concrete: Steel I-Beam Stringers or P.C. Girders	S/7.0	S/5.5
Concrete Box Girders	S/8.0	S/7.0

Exterior Beams:
a. Simple beam reaction, or
b. Load fraction = S/(4 + .25S)
(≥ 4 steel stringers; S = 6 – 14)

Special:
Spread Box Girders: Interior Load Fraction = $\frac{2N_L}{N_B} + k\frac{S}{L}$

Composite Box Girders: Load Fraction = $0.1 + 1.7R + \frac{0.85}{N_w}$

Method of Analysis

1. Orthotropic plate
2. Finite element or strip
3. Grillage analogy
4. Folded plate
5. Influence surfaces

Factors Affecting Design

1. Aspect ratio
2. Bridge stiffness parameter
3. Edge effects
4. Load position
5. Skew
6. Continuity
7. Diaphragms (type, location)

Specification Problems

1. Criteria format not consistent
2. Basis for criteria varies
3. Critical factors not considered
4. New bridge types require special studies
5. Loading conditions changed
6. Inconsistent safety factors
7. No criteria for rating

Current Design Practice

1. Timber deck/timber stringers
2. Concrete deck/steel I-beams
3. Concrete deck/P.C. girders
4. Steel grid decks/any stringer
5. Concrete deck/concrete T-beams
6. Segmental box girders
7. Concrete deck/spread box beams

Future Criteria

1. Load distribution criteria centralized
2. Simple criteria for "simple" bridges; Complex theories for "complex" bridges encouraged
3. Adaptable to all types of bridges
4. Separate design and rating criteria
5. Complete criteria for moment and shear

Produced by Office of Editorial Serv
Iowa State University, Ames, Iowa

Interaction Analysis of Asymmetric Sway Frames

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A novel method is presented for the approximate three-dimensional analysis of asymmetric sway frames subjected to torsional loading causing P-Delta effects.
The two most significant aspects of the new procedure are:

1. That actual structures need not individually be analysed on a rigorous elastic-plastic basis but by using their elastic buckling load and rigid plastic collapse load as reference parameters i e similar to the conventional in-plane analysis of single columns without the need for iterations.
2. That the proposed method can be used on a story-by-story basis for multi-story structures, thereby greatly reducing the number of variables compared with an investigation of the full frame. The load factor of the weakest story is then taken as the load factor applicable to the entire framework. More details and the principles of the new analysis technique are given in Refs.1-4.

The technique is suitable for three-dimensional frame structures made up of intersecting rectangular grids, ignoring local and torsional buckling of the members and disregarding their torsion and warping resistances.

The fundamental assumptions can be summarised as follows:

1. Any given frame structure can be grouped into a unique family of frames.
2. Each family of frames can be represented by a specific curve in a multicurve interaction graph.
3. The significant frames within a particular family of frames are a frame unaffected by P-Delta effects for which the failure load is equal to its rigid-plastic collapse load and a frame that fails completely elastically, i e failure is related to elastic buckling. The latter frame is termed the "limiting frame" of the frame family.
4. Between the two significant frames other frames can be located on the failure curve by reference to their ratio of elastic buckling load to rigid-plastic collapse load.

The presence of torsion is recognised by examining rigid-plastic collapse modes and elastic buckling modes in both directions of the rectangular frame grid and by elastically distributing the total applied lateral load to the individual frames on the grid when it comes to defining the geometry of the "limiting frame". The parameter $(0{,}4P_C/P_P)_\ell$ of the "limiting frame" is used to select the relevant curve from the multicurve diagram. The actual structure is then located on that curve by its ratio $0{,}4P_C/P_P$.

The establishment of the "limiting frame" is thus of prime importance when obtaining the failure load from the interaction graph. In its simplest format the ratio $(0{,}4P_C/P_P)_\ell$ of the "limiting frame" can be found from the equivalent ratio applicable to the actual structure, i e $0{,}4P_C/P_P$, by using Eq.(1) which is derived by equating elastic failure and first yield.

$$\left(\frac{0{,}4P_C}{P_P}\right)_\ell = \frac{0{,}4P_C}{P_P}\,\frac{Zf_P}{M}\,R \qquad \text{Eq.(1)}$$

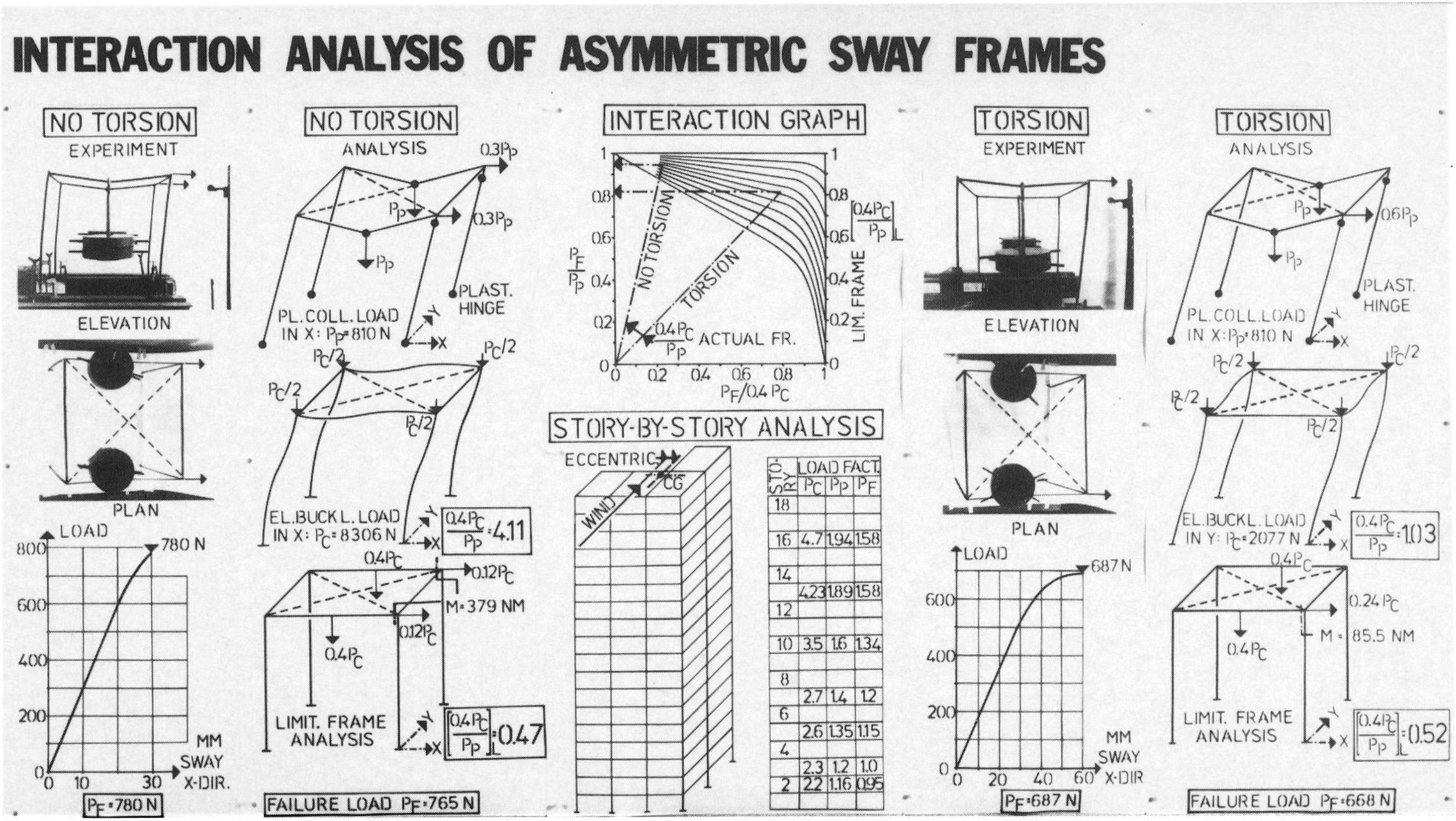
To solve Eq.(1) the actual structure is subjected to loading of the same configuration as the factored applied load but in magnitude related to the elastic buckling load of the frame. The parameter Z is the elastic section modulus, M=second-order elastic moment, f_P=stress at onset of yield. The factor R recognises that the reduction to the fully-plastic moment capacity of column sections due to axial load and bi-axial bending may be different for the actual and the "limiting frame". The value R can often be estimated, however, R=1 will mostly give satisfactory results. For the structure under consideration the lowest ratio $(0{,}4P_C/P_P)_\ell$ is significant.

The presented method compares well with experimental and rigorous analytical results. A single-story model framework subjected to torsion was recently tested by the author. The experimental failure load exceeded the predicted value by less then three per cent.

The shown multi-story structure is similar to a framework previously analysed by Hibbard and Adams(5). The lowest story load factor for proportionate vertical and horizontal loading is found as 0,95.

REFERENCES

1. Scholz, H "Evolution of an approx. analysis technique for unbraced steel frames", to be published in The Civil Engineer in South Africa
2. Scholz, H "Simplified interaction method for sway frames", Journal of Structural Division, ASCE, Vol 110, No.5, May 1984, pp992-1007
3. Scholz, H "A new multi-curve interaction method for the analysis of steel sway frames", Proc.3rd Intern.Colloquium on Stability of Metal Structures, Toronto, May 1983, pp431-448
4. Scholz, H "Interaction analysis of asymmetric sway subassemblages", to be published in Journal of Structural Division, ASCE, Vol.110, No.10, Oct.1984
5. Hibbard JR, Adams PF "Subassemblage technique for asymmetric structures", Journal of Structural Division, ASCE, Vol.99, ST11, Nov.1983, pp 2259-2268



INTERACTION ANALYSIS OF ASYMMETRIC SWAY FRAMES
NO TORSION
EXPERIMENT
ELEVATION
PLAN
LOAD
780 N
800
600
400
200
0
0 10 30
MM SWAY X-DIR.
PF = 780 N
NO TORSION
ANALYSIS
0.3PP
PP
0.3PP
PP
PLAST. HINGE
PL. COLL. LOAD IN X: PP = 810 N
PC/2
PC/2
PC/2
PC/2
EL. BUCKL. LOAD IN X: PC = 8306 N
0.4PC / PP = 4.11
0.4PC
0.12PC
M = 379 NM
0.12PC
0.4PC
LIMIT. FRAME ANALYSIS
[0.4PC / PP]L = 0.47
FAILURE LOAD PF = 765 N
INTERACTION GRAPH
PF / PP
NO TORSION
TORSION
0.4PC / PP
ACTUAL FR.
PF/0.4 PC
[0.4PC / PP]L
LIM. FRAME
0 0.2 0.4 0.6 0.8 1
STORY-BY-STORY ANALYSIS
ECCENTRIC
WIND
CG
STORY | LOAD FACT. PC | PP | PF
18
16 | 4.7 | 1.94 | 1.58
14
4.23 | 1.89 | 1.58
12
10 | 3.5 | 1.6 | 1.34
8
2.7 | 1.4 | 1.2
6
2.6 | 1.35 | 1.15
4
2.3 | 1.2 | 1.0
2 | 2.2 | 1.16 | 0.95
TORSION
EXPERIMENT
ELEVATION
PLAN
LOAD
687 N
600
400
200
0
0 20 40 60
MM SWAY X-DIR
PF = 687 N
TORSION
ANALYSIS
PP
0.6PP
PP
PLAST. HINGE
PL. COLL. LOAD IN X: PP = 810 N
PC/2
PC/2
PC/2
PC/2
EL. BUCKL. LOAD IN Y: PC = 2077 N
0.4PC / PP = 1.03
0.4PC
0.24 PC
M = 85.5 NM
0.4PC
LIMIT. FRAME ANALYSIS
[0.4PC / PP]L = 0.52
FAILURE LOAD PF = 668 N

Vibration Control of Stiffening Arch Bridge

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Ishikawa, Japan

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Kanazawa University
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A variety of technical problems related to highway bridges have been point out, because an increasing number of heavy trucks are seen on nation's highway in recent years, and the method of reinforcement of bridges has become the center of wide interest. In the reinforcement of bridges, it is necessary to consider the serviceability of bridges not only in terms of statistical and dynamical problems, but also the vibration felt by pedestrians.

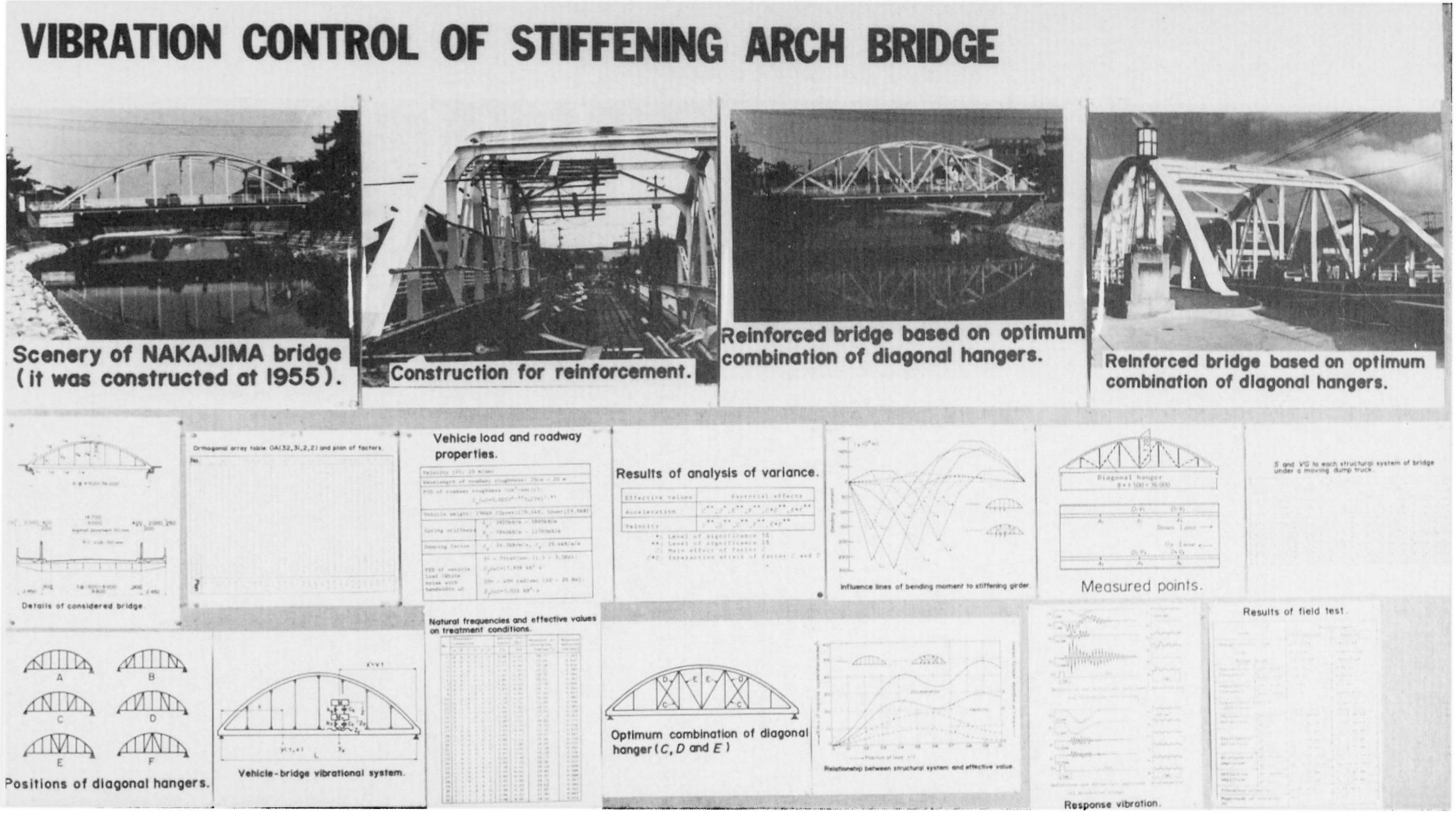
In this paper, a particular stiffening arch bridge (Lohse girder bridge), which holds these problems described above, is considered as case study. The method of reinforcement is investigated by the insertion of diagonal hangers. In order to find out the most efficient of reinforcement on this bridge, a statistical inference method (a design of experiments) is applied to this study. In this method, the evalution of vibration control is investigated. It is considered that the acceleration corresponds to the magnitude of vibration on the bridge and the velocity corresponds to the vibration felt by a pedestrian. Each effective value of the response acceleration and velocity is calculated by dynamic analysis of nonstationary response of the bridge with inserted diagonal hangers under a moving heavy vehicle, and the optimum combination of diagonal hangers is estimated from these effective values. The effect of insertion of the estimated optimum combination on the serviceability of this bridge based on the vibration sensibility of pedestrian, and the statistical and dynamic problems is investigated.

Using the calculated results described above, and taking aesthetic point into consideration, actual construction was done to reinforce the bridge. Before and after testing was done to determine the effect of the insertion of the diagonal hangers. From the measurd results of this field test, it could be seen that these results beared out the predictions of the analytical study.

The major conclusions of this study can be summarized as follows:

(1) The load carrying capacity of the stiffening girder increases because the applied load is disersed by these diagonal hangers.
(2) The excitation of the first asymmetric vibration is eliminated because the vibration mode is changed by the alteration of the bridge structure system and the natural frequency increases.
(3) The serviceability of this bridge is improved because the vibration felt by the pedestrian decreases.

Finally, from the results of this analytical study and field test, it is recognized that the method of reinforcement using diagonal hangers is a successfull way for vibration control in the stiffening arch bridge.


VIBRATION CONTROL OF STIFFENING ARCH BRIDGE
Scenery of NAKAJIMA bridge (it was constructed at 1955).
Construction for reinforcement.
Reinforced bridge based on optimum combination of diagonal hangers.
Reinforced bridge based on optimum combination of diagonal hangers.
Details of considered bridge.
Vehicle load and roadway properties.
Results of analysis of variance.
Influence lines of bending moment to stiffening girder.
Measured points.
Positions of diagonal hangers.
A
B
C
D
E
F
Vehicle-bridge vibrational system.
Natural frequencies and effective values on treatment conditions.
Optimum combination of diagonal hanger (C, D and E)
Relationship between structural system and effective value
Response vibration.
Results of field test.

Steel Bridge Girders, Cost Optimization

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Consulting Engineers and Planners
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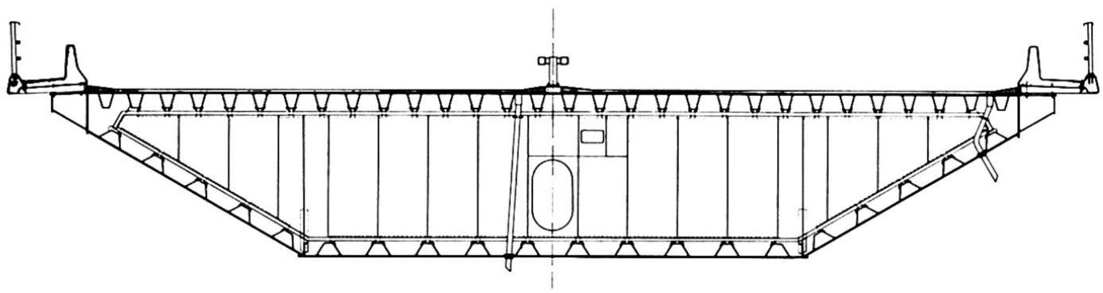
The steel box girder for the 3.3 km long bridge at Farø, Denmark has been made competitive by use of unusual design and construction methods.

A considerable saving has been possible by omission of painting of internal surfaces of the box girder, which amounts to more than 80% of the total steel surface. The corrosion protection of these surfaces is accomplished by ventilation by means of dehumidified air. The six dehumidification units represent low initial investment and are very economical in operation, each covering 5-600 m of bridge girder length. The external surface of the box girder to be painted has been reduced to a minimum by choice of a special cross section shape (refer to Farø bridge cross section below) with smooth exterior permitting an inexpensive initial painting cost and low maintenance.

The girder is composed of uniform steel panels welded by automatic welding, and a special assembly detail between exterior panels and diaphragms each 4 m has been detailed so as to require minimum of tight tolerance control during fabrication.

The box girder has been fabricated in a ship yard, all welded in full span sections each 80 m, and erected by simple lowering directly onto the pier tops. The girder continuity over full bridge length (1.6 km and 1.7 km) is subsequently established by field welding of box girders over the piers.

The bridge connection at Farø, which is part of European main highway E4, is presently under construction and is scheduled for completion Summer 1985.



Farø Bridge Cross Section

STEEL BRIDGE GIRDERS, COST OPTIMIZATION

Fabrication and maintenance costs for steel box girders may be optimized by:

1. Using identical or few types of similar panels.
2. Using simple panel connections for assembly of box cross sections.
3. Minimising the exterior surface area.
4. Corrosion protection of the box interior by dehumidification.

The steel box girders for the Farø Bridges were developed to satisfy the above criteria, and proved to be economically competitive in comparison with post-tensioned concrete box girders.

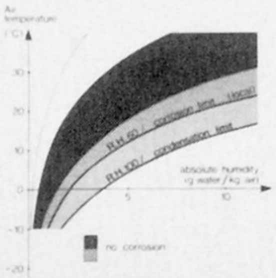
The internal corrosion protection scheme by circulation of dry air offers significant cost savings.

The dehumidification unit is based on the absorption principle and consist of readily available standard components. The steel surface is completely corrosion protected by relative humidities below 60%.

Dehumidification systems have been used in the Lille Bælt suspension bridge box girder since 1970, and the operating costs have been extremely low (approx. 1.5 US Cent per sq.m. per year). A similar installation will be used in the Farø Bridges, presently under construction in Denmark and scheduled for completion in 1985.

Owner: The Road Directorate, Denmark
Design and Supervision: Cowiconsult, Denmark
Fabrication and Erection: Monberg & Thorsen A/S, Denmark

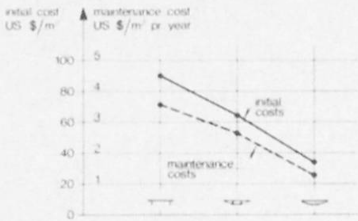
Relative humidity in box girder

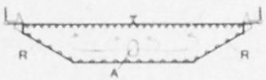


Corrosion protection of external and internal steel surfaces

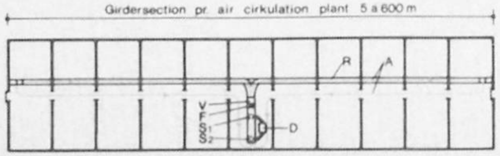
SURFACE AREAS	Girder types		
A_D deck area	20 m²/m	20 m²/m	20 m²/m
A_I external painted area	72 m²/m	50 m²/m	24 m²/m
A_I/A_D	3.6	2.5	1.2
Protection: external area	painted	painted	painted
internal area	–	dehumidified	dehumidified

COSTS FOR CORROSION PROTECTION PR m² DECK AREA

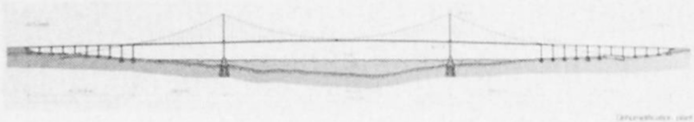




A : Air flow
R : Hollow stiffening ribs serving as air flow ducts
V : Fan
F : Air filter
D : Dehumidifying unit
S_1, S_2 : Fresh air/exhaust damper for ventilation



Lille Bælt Bridge 1970

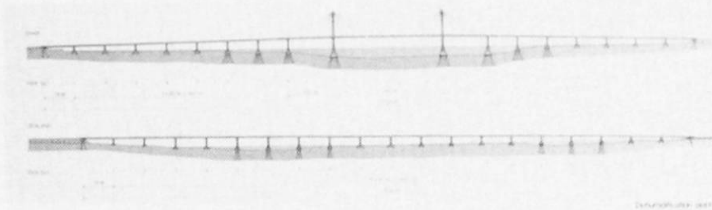


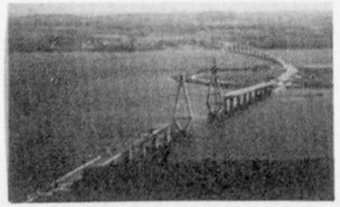
Steel box girder

Total length of girder	1,080 m
Total steel deck area	29,000 m²
Total steel weight in girder	12,000 t

Steel surface	Internal surface	External surfa[ce]
Area	200,000 m²	42,000 m²
Corrosion protection	Dehumidification plant	Paint

Farø Bridges 1985





Steel box girder

Total length of girder	3,326 m
Total steel deck area	64,000 m²
Total steel weight in girder	23,000 t

Steel surface	Internal surface	External steel surfa[ce]
Area	320,000 m²	76,000 m²
Corrosion protection	Dehumidification plant For each 500-600 m girder length	Paint
Initial cost	0.10 million US$	2.20 million US$
Maintenance cost	<0.01 million US$/year	0.08 million US$/year

Annacis Island Bridge

P.R. TAYLOR and **O.F. SIMONSEN**
CBA-Buckland and Taylor
Vancouver, BC, Canada

BRIDGE DESCRIPTION

Type of Bridge:	Modified Fan Cable Stayed Bridge.
Spans:	50m, 182.75m, 465m, 182.75m, 50m.
Tower Height:	154.3m above top of pilecap.
Midspan Shipping Clearance:	58.4m above High Water.
Traffic Capacity:	Initially 4 lanes of highway traffic.
Design Capacity:	6 lanes of highway traffic or 4 lanes plus 2 tracks for ALRT.

SUPERSTRUCTURE

The superstructure comprises a structural steel skeleton consisting of constant depth twin I beams and transverse floorbeams, which supports a composite precast concrete deck with a cast in place concrete overlay.

I beams:	2.1m deep by 18m long typically. Splices are bolted.
Floorbeams:	Tapered 1.6m to 1.8m deep by 27.2m long typical.
Floorbeam spacing:	4.5m typical.
Quantity of Structural Steel:	5,600 tonnes.
Grade of Structural Steel:	350 AT Category 2. 350MPa Yield Stress Atmospheric Corrosion Resistant Steel having a guaranteed Charpy Impact Strength of 27 Joules at -20°C.
Precast Deck Panels:	13.5m x 4.0m x 215mm typical - weight 35 tonnes approx.
Precast Concrete Strength:	55 MPa @ 56 days.
Overlay Concrete Strength:	55 MPa @ 56 days.

CABLES

Cables are Long Lay Galvanized Bridge Strand sheathed with black polyethylene. Every cable has a zinc filled cast steel socket at both ends. Cables terminate at tie beams in the towers where provision is made for jacking and adjustment.

Number of Cables:	192 main cables, 8 tie down cables.
Cable lengths:	49.5m to 237.5m.
Cable diameters:	80mm to 130mm.
Wire:	7mm diameter galvanized, U.T.S. 1520 MPa.
Cable Assembly Weights:	2 tonnes to 24 tonnes.
Total Cable Weight:	1505 tonnes (excluding sockets).
Total Socket Weight:	193 tonnes.

TOWERS AND BENTS (including Pilecaps)

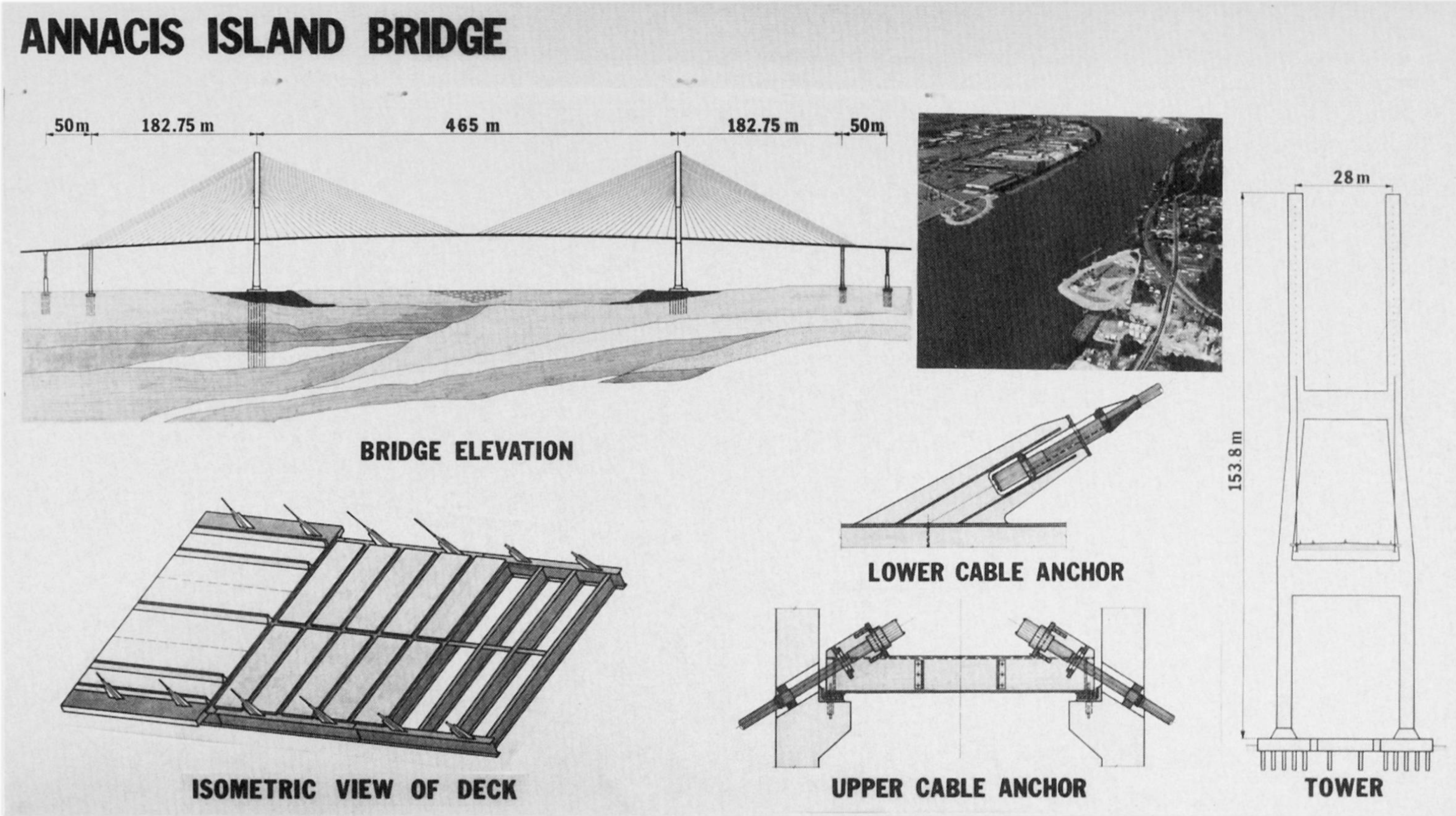
The towers and bents are reinforced concrete structures with provision for ductile behaviour in earthquake.

FOUNDATIONS

All of the foundations rest on steel piles. In addition, densification piles were used in the upper sands around piers N1, N2 and N3 to eliminate the possibility of liquefaction during earthquake.

SHIP IMPACT

Piers S1 and N1 have protective surrounds, designed to withstand the impact of a 60,000 DWT vessel travelling at 12 knots.


ANNACIS ISLAND BRIDGE
50m
182.75 m
465 m
182.75 m
50m
BRIDGE ELEVATION
28 m
153.8 m
TOWER
LOWER CABLE ANCHOR
ISOMETRIC VIEW OF DECK
UPPER CABLE ANCHOR

Hitsuishijima and Iwakurojima Bridges

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1. THE OUTLINE OF THE PROJECT

The Hitsuishijima and Iwakurojima Bridges with four-tracks railway(on lower deck) and four-lanes highway(on upper deck) are situated more or less midway along the Kojima-Sakaide route which forms the main project of the bridge activities between Honshu and Shikoku.
In early stage of the designing of these bridges, several bridge -types such as gerber truss(Fig. 1), cable-stayed bridge(Fig. 2) were considered. And cable-stayed bridges of Fig. 3 and Photo. 1 were finally chosen by considering the navigative, constructional and economical requirements and also from aesthetical points of view.
Work on this project was started in Oct. 1978 with the substructures. Construction of the bridges are scheduled to be complete in March 1988.

2. DESIGN CONDITION

Highway(upper deck) :	Four lane	Design spead: 100Km/h
Railroad(lower deck):	Two ordinary lines	Design spead: 120Km/h
	Two Shinkansen lines	Design spead: 160Km/h

3. SUMMARY OF THE SUPERSTRUCTURE

(1) Span length: 185+420+185m, (2) Main truss: Warren truss with vertical members(high 13.9m, width 27.5m), (3) Cable: Multi-stay cable system(parallel wire strand using 7mmϕ steel wire) anchoring to HiAm sockets, (4) Tower: Rigid steel frame type(height 136m), which has image of Japanese traditional form such as Japanese Helmet, (5) Shoe: Spring shoes(Fig. 6) with corned disc springs(Photo. 2)(700mmO.D., 350mmI.D., 32mmT) are installed at the truss ends to control the longitudinal deformation response due to earthquake, (6) Shock absorber: Solid rubbers are also installed on the both end piers to control the extreme seismic ampritudes of the truss and to protect the expansion joints of tracks, (7) Joint: Friction type joints using high strength bolts are used to connect the truss panel points and the pylon.

HITSUISHIJIMA AND IWAKUROJIMA BRIDGES

Fig. Gerber Truss Typ

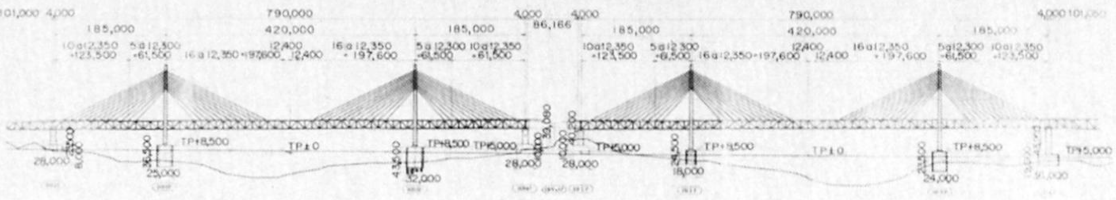

Fig. 3 General Profile

Fig. 5 Results of Dynamic Analysis

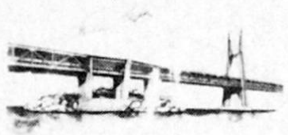

Fig. 2 Cable-Stayed Type

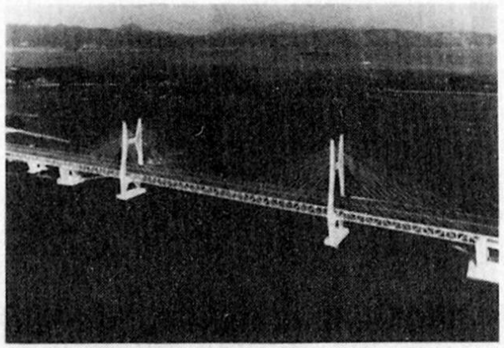

Photo. 1 Aerial View of Hitsuishijima Bridge (photo-montage)

Fig. 6 Spring Shoe

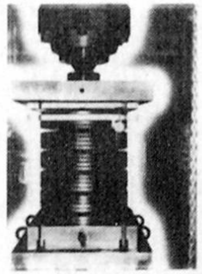

Photo. 2 Coned Disc Springs for Static and Dynamic Loading Experiments

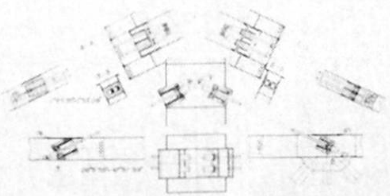

Fig. 4 Cable Connections

Extreme Span Suspension Bridges – Structural Systems

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With increasing spans the self weight of the main cables of a suspension bridge plays a more and more dominant role in relation to the total load. Thus, it is essential to choose a sag/span ratio near the quantatively optimized value in order to minimize the amount of cable. This value is generally considerably higher than the value chosen to give adequate stiffness /1/.

In general considerations regarding total cost and deflections will lead to opposite requirements to the sag/span ratio. In the research project described in /2/ it is shown that this effect becomes even more pronounced for extreme spans, such as the 3000 m span investigated. Thus, the traditional way of reducing the deflections through choice of a smaller sag will, in this case, significantly affect the total economy.

The present investigation deals with the problem of improving the deformational characteristics of suspension bridges with extreme spans by modifying the conventional structural system.

The investigation shows that a system with a longitudinally fixed stiffening girder and a central node clamping the main cable to the girder at midspan and having a sag/span ratio of 1:9 will give the same deflection under the critical asymmetric load as a conventional system having a sag/span ratio of 1:12. This leads to a saving in the total amount of steel of approximately 100,000 tons (measured as the equivalent quantity of structural steel), corresponding to saving in the magnitude of 250 million dollars for a bridge with a 3000 m main span. Compared to this saving, the cost of clamping the main cable appears to be negligible.

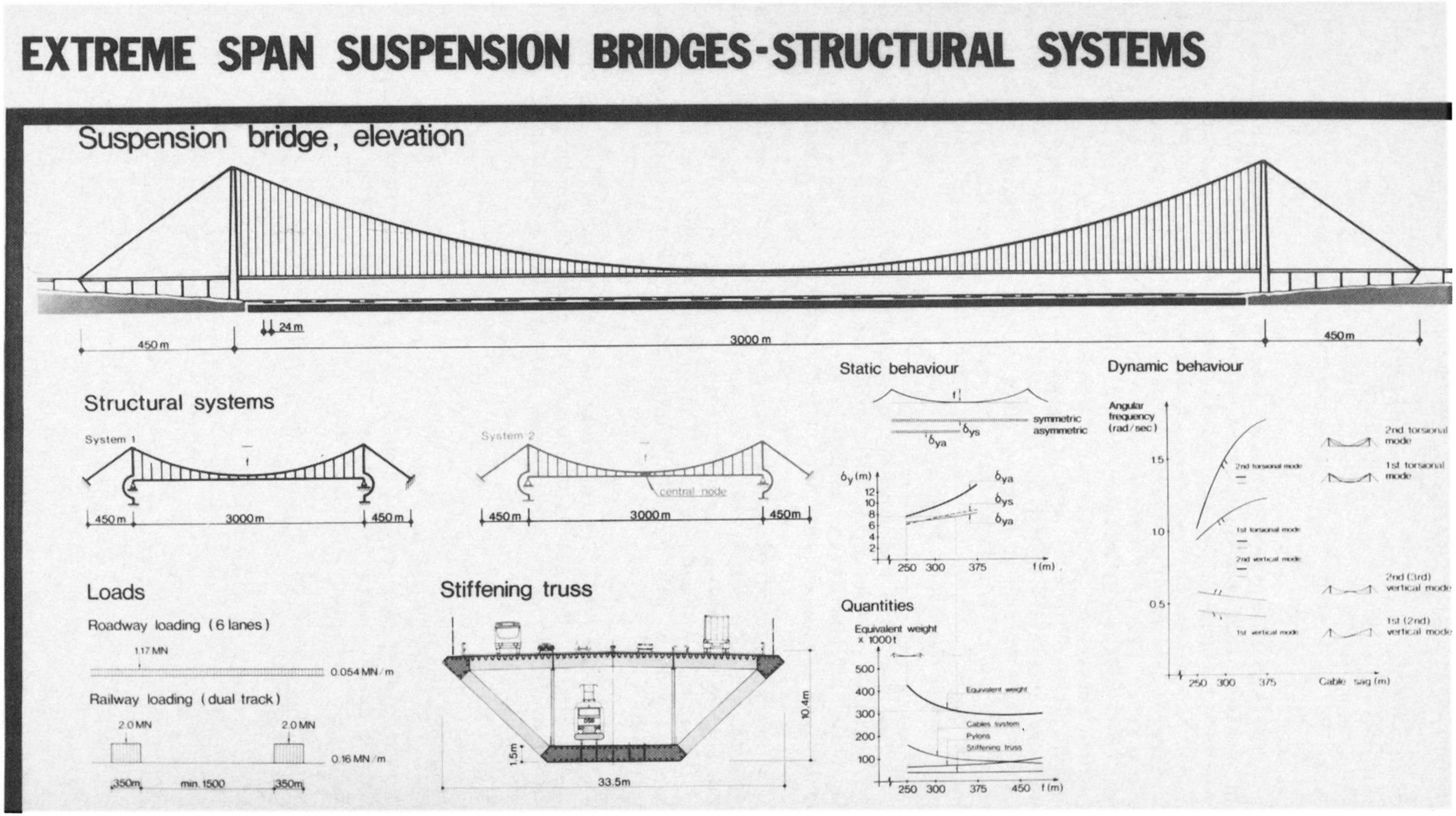
In the investigation it is also shown that the ratio between the torsional and the vertical frequencies will increase with increasing sag, thus improving the resistance against flutter.

The present investigation forms part of a research project on bridges with extreme spans /2/. The project has been sponsored by the Cowi Foundation and carried out at the Technical University of Denmark in collaboration with Cowiconsult, Consulting Engineers and Planners, Copenhagen.

REFERENCES

/1/ Niels J. Gimsing: Cable Supported Bridges, Concept & Design, Wiley 1983.

/2/ Niels J. Gimsing, Anders Borregaard Sørensen: Investigations into the Possibilities of Constructing Bridges with a Free Span of 3000 m. Report No. 168, Dept. of Structural Engineering, Technical University of Denmark.


EXTREME SPAN SUSPENSION BRIDGES-STRUCTURAL SYSTEMS
Suspension bridge, elevation
24 m
450 m
3000 m
450 m
Structural systems
System 1
450 m
3000 m
450 m
System 2
central node
450 m
3000 m
450 m
Loads
Roadway loading (6 lanes)
1.17 MN
0.054 MN/m
Railway loading (dual track)
2.0 MN
2.0 MN
0.16 MN/m
350m
min. 1500
350m
Stiffening truss
10.4m
1.5m
33.5m
Static behaviour
symmetric
asymmetric
250
300
375
f(m)
Quantities
Equivalent weight x 1000 t
500
400
300
200
100
Equivalent weight
Cables system
Pylons
Stiffening truss
250
300
375
450
f(m)
Dynamic behaviour
Angular frequency (rad/sec)
1.5
1.0
0.5
2nd torsional mode
1st torsional mode
2nd vertical mode
1st vertical mode
2nd (3rd) vertical mode
1st (2nd) vertical mode
250
300
375
Cable sag (m)

Dynamic Loading of Highway Bridges; Ontario

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The Ontario Highway Bridge Design Code (OHBDC) contains provisions for vehicle load and associated dynamic load and vibration which differ from other codes. The provisions base the design truck load and design lane load on load surveys carried out in Ontario. These design loads lead to legal loads and overload control. With such carefully selected design loads which are representative of actual traffic loads, it is essential that the additional allowance for the dynamic effects of load are also representative of actual vehicle-bridge response. The provisions for dynamic load allowance (impact) still consider that the dynamic effects of vehicles crossing highway bridges can be described in terms of an equivalent static effect that is a fraction of the design vehicle load. The magnitude of this effect depends upon the governing load, e.g., axle or design truck, and may also depend upon the natural frequency of the structure rather than span length.

Few codes are based on a limit states philosophy for both design and evaluation. Accordingly, new provisions were required for OHBDC which represent adequately the random effects of the dynamic component of load as typical design and evaluation vehicles traverse a span.

The results of the tests are presented and described in the context of a design code for highway bridges. Some existing code provisions were found unconservative for structures having a first flexural frequency lying between 2.0 and 5.0 Hz. Calibration of the load factors for dynamic load allowance for a reliability based limit states design code is described (1).

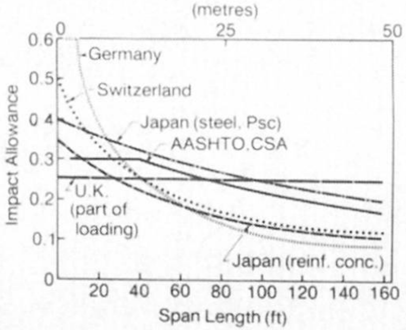
In summary, the dynamic response of modern bridges to modern vehicles is described. Provisions as to how this response might be catered for in a design code that represents the significant mechanism of vehicle-bridge interaction are given.

Reference

1. "Ontario Highway Bridge Design Code and Commentary", Highway Engineering Division, Ontario Ministry of Transportation and Communications, Downsview, Ontario, Canada, 1983.

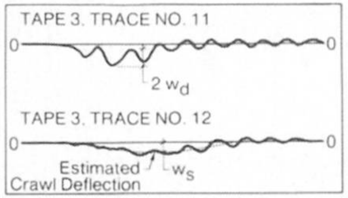
DYNAMIC LOADING OF HIGHWAY BRIDGES; ONTARIO

THE SITUATION
New Limit States Design Code in Ontario (1979, Revised 1983).
Prior to 1979, Legal Loads were greater than Design Loads, and Observed Dynamic Effects

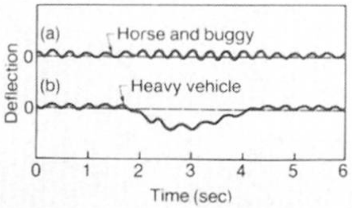


were greater than Design Effects. Existing codes indicate wide differences in impact values.

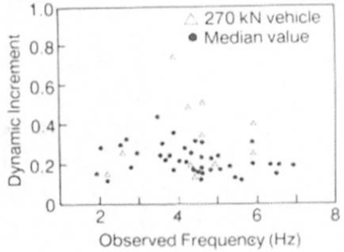
Dynamic effects are superimposed on a static, crawl deflection curve (below) so amplifying the response to static load. Increases from the static value of 30 to 40 percent are not uncommon.



A horse and buggy can excite a structure to a greater extent than a heavy vehicle.

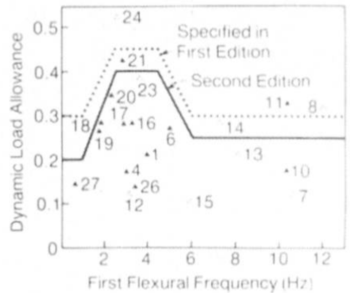


Typical field data for dynamic amplification indicate considerable scatter due to variation in vehicle type and pavement roughness. Mean amplification tends to increase with speed and to increase with bridge frequency in the range 2 to 5 Hz - typical of the bounce frequency of modern heavy vehicles (below). The results lend themselves to a statistical treatment.



__Dynamic load allowance__ is a fraction of highway live load to cater for dynamic effects of vehicle & bridge, and riding surface irregularity.

The **DLA** for Ontario bridges is given below for both the 1979 and 1983 provisions. The envelope of observed values scaled according to the calibration process corresponds to the provisions. Data obtained in Switzerland also confirms DLA increase in the 2 to 5 Hz range.



DLA Values

Load	**Value**
Axle/wheel -	0.40
Two or more axles - (span less than 22 m)	0.30
Truck - (A function of frequency)	0.40 to 0.20
Lane load, UDL -	0.10
Vehicles with speed control (10 km/h) -	30% of above

DLA Values

Field Inspection of Experimental Timber Bridges

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In the summer of 1983, a unique inspection study was made of 18 experimental timber bridges in the U.S. National Forests. Constructed in the late 1960's and early 1970's, the various timber bridges contained novel features expected to improve performance. The bridges were built in various national forests in seven states and varied in length from 20 to 168 feet (20 to 73 feet individual spans). The number of spans ranged from one to four. Primarily, they were constructed with transverse glued-laminated (glulam) panel decks and a variety of interpanel connections. Some bridges had nail-laminated (nail-lam) decks for comparative purposes. Also, different types of members, construction and materials were used in the remainder of the superstructure and substructure. Preparation and installation of the experimental features was coordinated with the U. S. Forest Products Laboratory in Madison, Wisconsin. The objective of the study was two-fold: (1) to determine the in-place performance of timber bridges, especially of glued-laminated panel decks, and (2) to determine patterns of moisture content in order to assess the merits of dry-use versus wet-use design stresses. On average, about 100 moisture content readings were taken per bridge.

Overall, the inspected bridges were in excellent structural condition. Glulam decks generally provided a more effective roof over stringers than nail-lam decks but both types had high moisture content. In contrast, the stringers were relatively dry. Stringer readings in excess of 20% were infrequent by the average moisture content in both decking types exceeded 20%. For bottom zones of stringers, it appears likely the moisture content would generally remain well below 20%. Readings above 30% were rare in all components except nail-lam deck. The observations about moisture content strongly suggest modern timber bridges components remain below fiber saturation condition for at least 20 years.

Moisture content data support the use of dry-use stresses for bottom laminations of glulam stringers for at least a 20-year service life. Readings between 13% and 15% were the norm for glulam and although occasional values above 16% were found, the soundness of the material appeared invariant. Except near abutments, dry use stresses for top laminations are similarly justified. Dry use stresses for solid-sawn timber are also supported by the findings in this study. Virtually all readings were at or below 19%, including in the abutment zone. Conversely, the observations do not support the application of dry-use stresses to any decking regardless of treatment method.

Typically, roadway conditions were excellent, providing for smooth passage regardless of surfacing. There was extensive asphalt cracking only where the surface was unusually thin. Evidence of deterioration either due to propagation of cracks or presence of potholes was rare. Dowel-connected deck panels were tightly mated.

A mistake was done here while photographying the poster:
the photography is in fact unavailable.
We regret for any inconvenience and are very sorry
for the authors of this poster.

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Univ. Polit. de Barcelona, Barcelona, Spain

TOWARDS A UNIFIED COMPREHENSIVE SYSTEM IN DESIGN OF REINFORCED AND PRESTRESSED STRUCTURES

1.- INTRODUCTION

- LIMIT-STATE METHOD APPEARS ALREADY IN MANY REINFORCED CONCRETE STRUCTURES CODES AS A PHILOSOPHY OF DESIGN.
- MORE RECENTLY, PRESTRESSED CONCRETE STRUCTURES CODES HAVE ADOPTED THIS DESIGN PHILOSOPHY.
- HOWEVER, CURRENT DESIGN PROCEDURES MAKE A SEPARATED TREATMENT OF REINFORCED AND PRESTRESSED CONCRETE STRUCTURES.
- WHAT ADVANTAGES ARE THERE A PRIORI IN A UNIFIED DESIGN TREATMENT?
  - MORE CONSISTENCY IN CODES.
  - SYNTHETIZATION FOR TEACHING PURPOSES.
  - OPENING OF A NEW PERSPECTIVE FOR FUTURE ACHIEVEMENTS IN THE CONCEPTION AND DESIGN OF CONCRETE STRUCTURES.
- THE IDEAS DEVELOPED HERE ARE INCLUDED IN A TREND OF RESEARCH IN WHICH REINFORCED CONCRETE COULD BE UNDERSTOOD IN THE FUTURE AS A SINGULAR AND LIMIT CASE OF PRESTRESSED CONCRETE, BEING $P=0$.

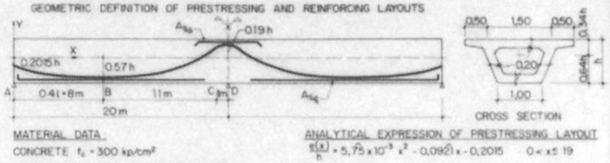
2.- METHODOLOGY

IN PRINCIPLE ONLY CONCRETE STRUCTURES COMPOSED BY LINEAR ELEMENTS WILL BE TREATED HERE.

- CROSS-SECTIONAL SHAPE IS FIXED A PRIORI IN DESIGN, DEPENDING MAINLY ON ECONOMICAL CRITERIA (i.e., CONSTRUCTION) RELATED TO BOTH REINFORCED AND PRESTRESSED CONCRETE TECHNIQUES AND OTHER GENERAL STRUCTURAL AND FUNCTIONAL FACTORS.
- IN ADDITION, ANOTHER PRIOR PARAMETER CAN BE THE RELATIVE POSITION OF LONGITUDINAL STEEL INTO CROSS-SECTIONAL DEPTH OR, BETTER, THE SHAPE OF REINFORCEMENT AND PRESTRESSING LAYOUTS.
- FURTHERMORE, THERE IS AN SPATIAL CORRELATION BETWEEN PRESTRESSING FORCE P VALUES (FRICTION LOSSES) DEPENDING ON THE CONTINUITY OF TENDONS.
- STARTING FROM THAT THERE IS A SET OF DESIGN PARAMETERS:
  - h, CROSS-SECTIONAL DEPTH, CONSTANT OR VARIABLE (ALSO A PRIORI DECISSION IN GENERAL) ALONG THE ELEMENT.
  - A_s, MAIN LONGITUDINAL REINFORCEMENT AREA.
  - A_{st}, TRANSVERSAL REINFORCEMENT AREA.
  - A_p, PRESTRESSING STEEL AREA ($A_p=0$, IN R.C.)
  - P, PRESTRESSING FORCE ($P=0$, IN R.C.)
  - b_w, WEB WIDTH.
  - b, FLANGE WIDTH.
  - ......
- AMONG THESE PARAMETERS, IT SEEMS USEFUL TO OUTLINE h, A_s, A_p AND P.

3.- EXAMPLE

GEOMETRIC DEFINITION OF PRESTRESSING AND REINFORCING LAYOUTS



MATERIAL DATA:

CONCRETE $f_c = 300$ kp/cm²

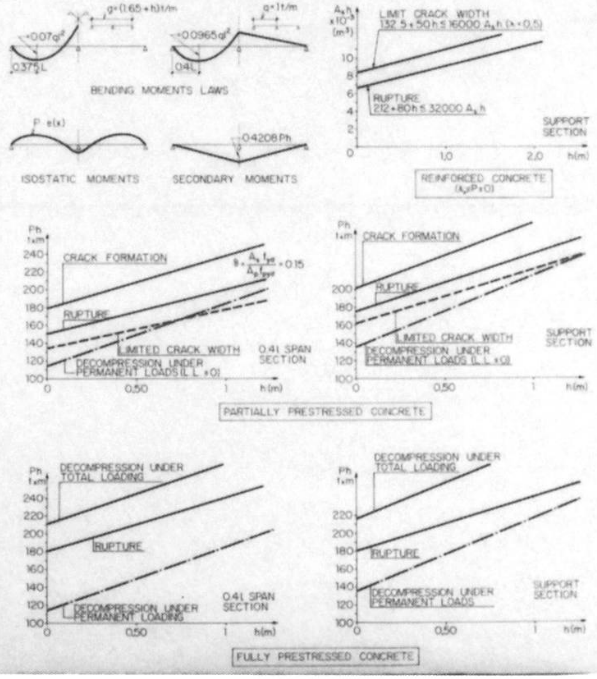
REINFORCING STEEL $f_{sy} = 4600$ kp/cm²

PRESTRESSING STEEL $f_{py} = 17000$ kp/cm²; $f_{p,max} = 18800$ kp/cm²

ANALYTICAL EXPRESSION OF PRESTRESSING LAYOUT

$$\frac{e(x)}{h} = 5.7\hat{5}\times10^{-3}x^2 - 0.092\hat{1}x - 0.2015 \qquad 0 < x \le 19$$

$$\frac{e(x)}{h} = -0.06\hat{3}x^2 + 2.5\hat{3}x - 25.14\hat{3} \qquad 19 < x \le 20$$



2.- (Cont.)

DIRECTLY LINKED TO BENDING, AS THE MOST SUITABLE REGARDING THIS WORK.

- ON THE OTHER HAND, A NUMBER OF DESIGN CRITERIA, SUCH AS ECONOMY, DURABILITY, AESTHETICS AND, IN GENERAL, THE LIMIT STATES MUST BE SATISFIED.
- LIMIT STATES VARY FROM ONE CODE TO OTHER. IN GENERAL THE FOLLOWING CAN BE INCLUDED:
  - ULTIMATE LIMIT STATES:
    - EQUILIBRIUM, RUPTURE (BENDING, SHEAR, etc.), BUCKLING, ...
  - SERVICIABILITY LIMIT STATES:
    - DEFORMABILITY, CRACKING, VIBRATIONS, ...
- AMONG THESE, EMPHASYS WILL BE MADE ON THE LIMIT STATES RELATED TO BENDING SUCH AS FLEXURAL RUPTURE, DEFORMABILITY AND, IN PARTICULAR, THE DIFFERENT LEVELS OF CRACKING CONTROL (DECOMPRESSION, CRACK FORMATION AND LIMITED CRACK WIDTH) GOVERNING THE DEGREE OF PRESTRESSING (TOTAL PRESTRESSING, PARTIAL PRESTRESSING AND, IN THE LIMIT, BEING $P=0$, R.C.).
- THE GENERAL CONDITIONS ABOVE MENTIONED ARE LINKED TO THE DESIGN PARAMETERS THROUGH A NUMBER OF RELATIONS USED IN THE ANALYSIS, REFERRED TO THE CRITICAL SECTIONS, SUCH AS:
  - RUPTURE $\quad h\,(K_1 A_s f_{yd} + K_2 A_p f_{pyd}) \geq K_3 + K_4\, Ph + K_5\, h$
  - DEFORMABILITY $\quad \alpha f(h^3) \geq \beta + \gamma\, Ph + \delta\, h$
  - CRACKING $\quad$ DIFFERENT EXPRESSIONS MUST BE USED FOR THE UNCRACKED STATE (IN TERMS OF STRESSES) AND FOR THE CRACKED (IN TERMS OF CRACK WIDTH). A GENERAL EXPRESSION COVERING THE WHOLE FIELD CAN BE:
    $\lambda h (K_1 A_s f_{yd} + K_2 A_p f_{pyd}) \geq K_6 + K_7\, P h + K_8\, h$
    IN WHICH λ SHOULD BE ADJUSTED TO COVER ALL POSSIBLE SITUATIONS IN THE CRACKING LIMIT STATES.
- OTHER GENERAL RELATION CAN BE STATED: $P = K\, A_p\, f_{pyd}$

NOTES: TERM Ph INCLUDES PRESTRESSING SECONDARY EFFECT.
THE VALUES OF THESE DESIGN PARAMETERS ARE FINALLY DETERMINED ACCORDING TO ECONOMICAL CRITERIA, FIXING IN GENERAL THEIR MINIMUM VALUES COMPATIBLE WITH THE DESIGN CONDITIONS.
IN THE CASE OF R.C. THE SIMPLICITY OF THE PROBLEM ALLOWS TO ADD ECONOMICAL CONDITIONS EXPLICITELY (i.e. RELATION SHIP BETWEEN h AND A_s FOR A MINIMUM COST THAT PROVIDE A SUFFICIENT ULTIMATE MOMENT).

4.- CONCLUSIONS

- A UNIFIED METHOD FOR THE DESIGN OF REINFORCED AND PRESTRESSED CONCRETE STRUCTURES, BASED IN A JOINT DEFINITION OF RELATED PARAMETERS (i.e. h, A_s, A_p, P, GOVERNING BENDING) HAS BEEN STABLISHED STARTING FROM LIMIT STATES CONDITIONS.
- THE RESULTS OF THE EXAMPLE ANALYSED ACCORDING TO THAT METHOD ARE LOGICAL, BEING THE MOST REMARKABLE ASPECTS:
  - ONCE THE LIMIT STATES THAT MUST BE SATISFIED ARE PREDEFINED, DESIGN PARAMETERS CAN BE OBTAINED THROUGH THE CONDITIONS ABOVE EXPRESSED.
  - THE MORE RESTRICTIVE LIMIT STATES IN THE DESIGN CAN BE IDENTIFIED AS A FUNCTION OF THE STRUCTURAL TYPE, GEOMETRY, LOADING AND DEGREE OF PRESTRESSING (FROM R.C. TO FULLY P.C.) SO THAT THE REMINDER LIMIT STATES ARE ONLY FOR VERIFICATION.
  - FACTOR λ PLAYS AN IMPORTANT RULE IN THIS TREATMENT. A GOOD DESIGN REQUIRES AN ADEQUATE ELECTION OF λ.
- FURTHER RESEARCH IS NEEDED IN ORDER TO SET THE RANGE OF VALUES OF λ ASSOCIATED TO A SPECIFIC LIMIT STATE FOR DIFFERENT CROSS-SECTIONS AND DEGREES OF PRESTRESSING.

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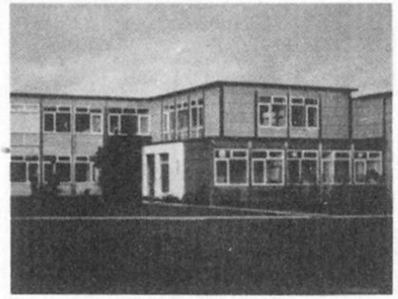
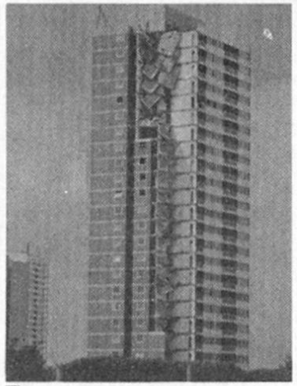
STRUCTURAL SAFETY OF BUILDINGS - TODAY AND TOMORROW

TODAY - *Most building structures are **SAFE** and **SERVICEABLE** for their required life*

A few failures do occur

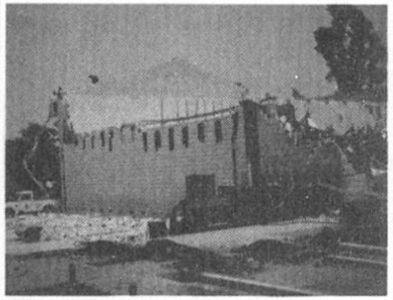


Pre-cast concrete System Construction
Degradation of components following corrosion of steel



Pre-cast concrete Panel Structure
Partial collapse following a gas explosion

- much can be learnt from them



Timber Trussed-rafter Longer-span roof
Collapse due to lack of bracing

Large population of similar buildings required inspection and remedial action

TOMORROW - *Failures - only a few today - can be fewer tomorrow*

Defence strategies to control stability

- *Explicit design choice of one or more of:*
  - *Multiple independent load paths.*
  - *Devices to allow structure to avoid carrying load.*
  - *Local strength increases to enhance overall strength.*
  - *Environmental and performance monitoring and control systems.*

Populations of similar structures

- *Design so that failure is first manifest on a local scale and will inhibit use.*
- *Structures should be robust, and should provide feedback signals to the user of damage, overloading or local degradation.*

Buildings with Long-span roofs

- *Use more stringent structural design criteria than for normal buildings.*
- *Exercise tighter control and checks of design and construction, to reduce the risk of design faults or of construction outside specification.*

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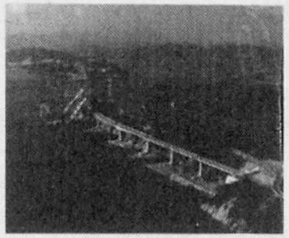
Honshu-Shikoku Bridge Authority, Tokyo, Japan

AERODYNAMIC STABILITY OF LONG-SPAN BOX GIRDER

Tozaki Bridge is the north approach to Ohnaruto Br. (suspension bridge) of Kobe-Naruto Route, Honshu-Shikoku Bridge Project, Japan.

Tozaki Bridge is 2 sets of continuous box girders and has slender configuration with large overhanging brackets.



The bridge was to be constructed along a steep ridge called Tozaki which faces the Pacific Ocean.

Thus, strong wind is expected so that the basic wind speed which is defined as 10 minutes average speed expected to occur once or twice in 150 years is decided to be 50m/s.

And, bad influence by the topography was also anxious.

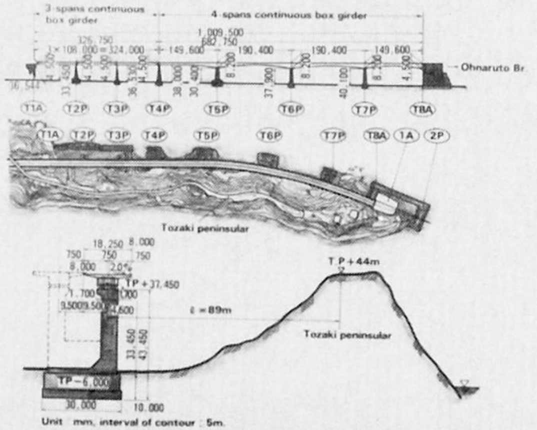


Fig-1 General view of Tozaki B Bridge

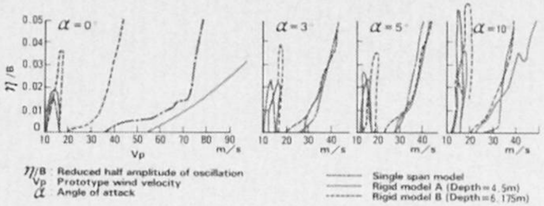
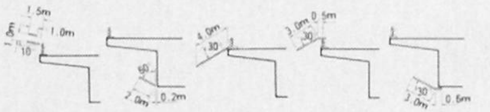


Fig-2 Response of original section (without topography)

The original section was suspected to have unstable aerodynamic behavior, so, wind tunnel tests were conducted to reveal that the original section had vortex-induced oscillation at the wind speed of 15 to 20 m/s and galloping which should not occur below the speed of 92 m/s (10minutes average), the dynamic design wind speed.

Various stabilizers and their effects were tested in wind tunnels.



Stabilizer		Double flaps		Lower skirt		Upper skirt		Upper deflector		Lower deflector	
Model		A	B	A	B	A	B	A	B	A	B
Amplitude of vortex-induced oscillation(mm)	$\alpha=3°$	0	0	520	770	–	1,210	520	–	670	–
	5°	0	0	680	930	–	470	640	–	670	–
	7°	0	0	880	1,100	–	–	770	–	670	–
Emergence velocity of galloping(m/s)	$\alpha=3°$	>100	82	>100	34	–	90	>100	–	>100	–
	5°	76	55	83	37	–	65	93	–	86	–
	7°	44	37	84	46	–	–	87	–	69	–

Table-1 Various stabilizers and their effects (rigid models)

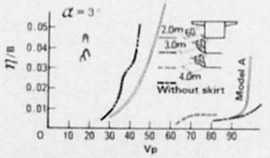


Fig-3 Effect of lower skirt (model B)

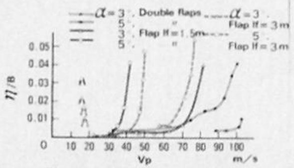


Fig-4 Effect of flap (model A)

As the results of a series of wind tunnel testings, the final stabilizers were determined as follows;

Double flaps were attached to the whole length of the bridge in order to restrict the amplitude of vortex-induced oscillation less than 200 mm ($\eta/B=0.01$) which was the allowable value from the viewpoint of fatigue and runnability of automobiles.

And, intermittent lower skirts to suppress galloping which might destroy the bridge in a short time if it appeared were provided to the 4-spans continuous girder.

After the completion of the bridge in the summer of 1983, oscillation and wind have been observed, but no considerable oscillation has been generated so far, because no strong wind beyond 20 m/s has unfortunately blown.

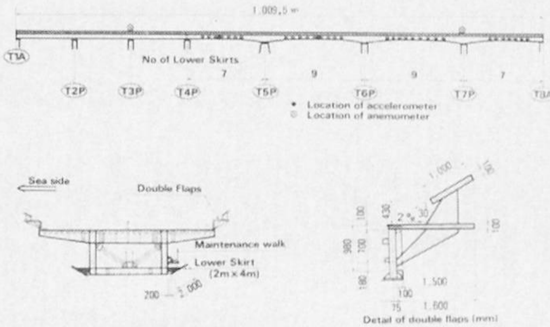


Fig-5 Actually adopted stabilizers and location of sensers

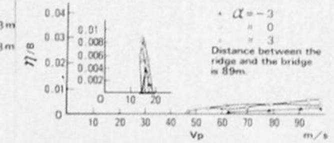


Fig-6 Response of improved section (single span model with topography)

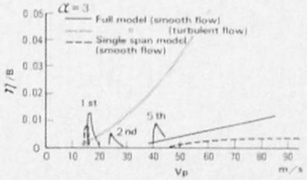


Fig-7 Response of improved section (elastic models in smooth or turbulent flow)