Zeitschrift: IABSE congress report = Rapport du congrès AIPC = IVBH Kongressbericht

Band: 12 (1984)

Rubrik: Poster session 6: Concrete structures

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POSTER SESSION 6

Concrete Structures

Structures en béton

Betontragwerke

Coordinator: R.S. Stilwell, Canada

New Column for Tall Reinforced Concrete Buildings

Kiyoshi MUTO, Kuniaki SATO, Satoshi BESSHO
Minoru FUKUSHIMA et al.
Kajima Corporation
Tokyo, Japan

A twenty-five story condominium constructed by Kajima Corp. is the highest reinforced concrete building in Japan, meeting the structural requirements for earthquake resistance. However, demand for the construction of taller buildings, particularly in urban reconstruction, is increasing. Therefore it has become necessary to develop new structural systems, especially a column system applicable to high-rise buildings taking into account earthquake resistance, economy and execution of works.

Now, in the twenty-five story condominium, the special lateral reinforcement which combines a circular spiral hoop and conventional lateral hoop is used. Since the concrete is effectively restrained by this special lateral reinforcement, it was proved that the restoring force is slightly reduced by cyclic lateral loading as shown in Fig.2.

So, for 30～40 story buildings, we have devised a new column system using high-strength concrete, small-size H-shaped steel and special lateral reinforcements, as shown in Fig.1.

And many specimens were tested to investigate the structural characteristics of the column system. The central compressive force was applied to test specimens as shown in Fig.3. As a result of this test, the maximum load capacity and its reduction ratio for the KS-type column compared to the central compressive force are the same or more than those for the conventional H-type column which uses H-shaped steel with a large sectional area.

The bending and shearing force was applied to test specimens as shown in Fig.4. As a result of these tests, the special lateral reinforcement is superior to the conventional lateral reinforcement on the capacity of restoring force after the maximum load. And the load capacity and ductility of this column are sufficient performance comparing with the conventional column under high compressive or tensile axial load. In addition, this new column has restoring characteristics regardless of the internal H-shaped steel load direction.

As a result of our research, design range of the column has been setting up like Fig.5. And the construction of economical forty-story buildings has been made possible.

NEW COLUMN FOR TALL REINFORCED CONCRETE BUILDINGS

BACKGROUND

In Japan, demand for the construction of 30~40 story condominium is growing. So it has become necessary to develop new column system taking into account earthquake resistance, economy and execution of works.

Fig. 1 Newly Devised Column

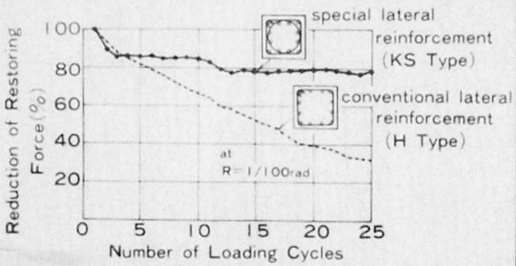


Fig.2
Effects of Special Lateral Reinforcements on Restoring Force for Cyclic Loading

RESEARCH AND DEVELOPMENT

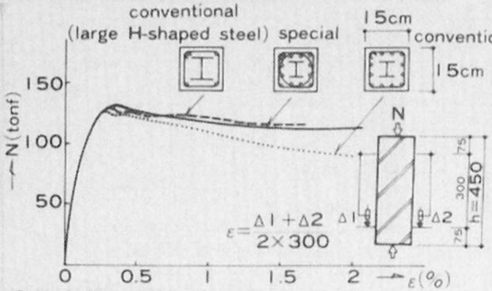
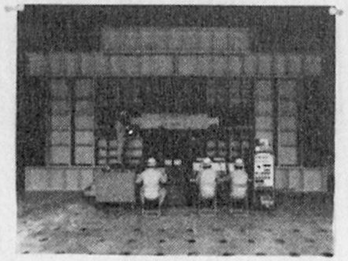
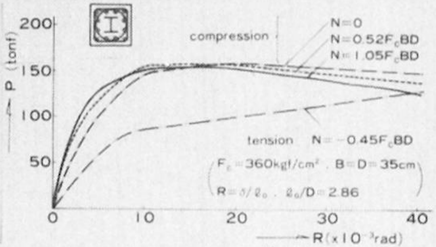
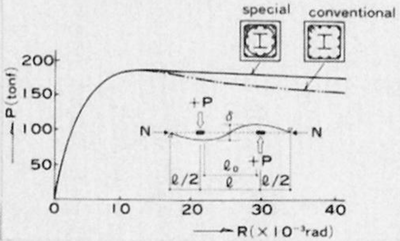


Fig.3 Axial Loading Tests







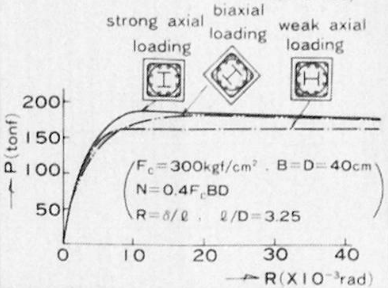


Fig.4 Bending and Shear Tests under Constant Axial Load

DESIGN

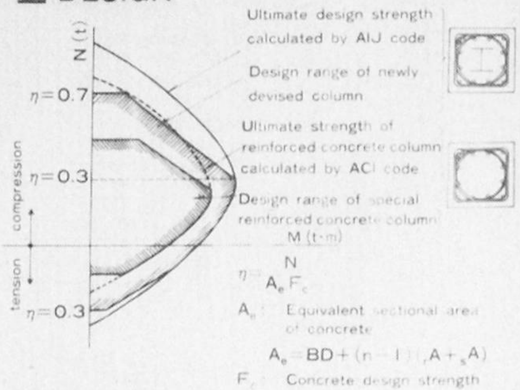
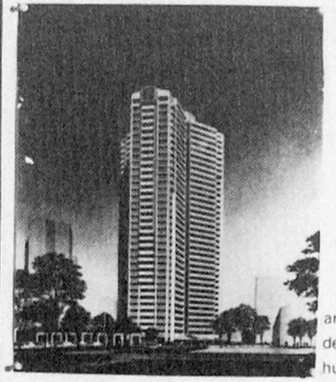


Fig.5 Design Range of Column



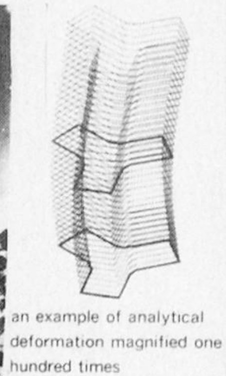


Fig.6 Perspective of First Building in which System to be Applied

Cable Stays for Bridges

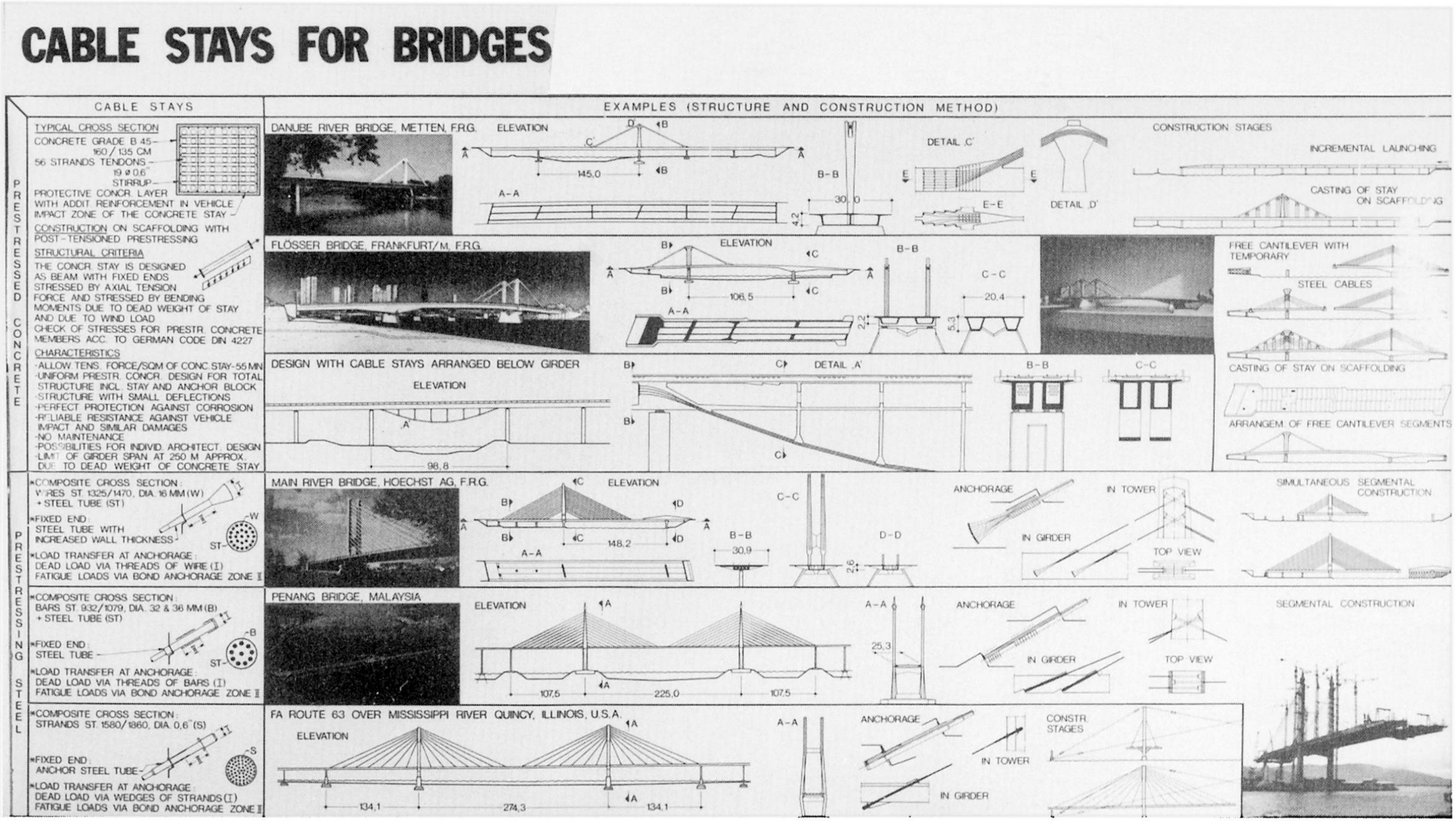
H. SCHAMBECK
Civil Eng.
Dyckerhoff & Widmann
München, FR Germany

DYWIDAG POSTTENSIONING SYSTEMS

Type of steel	Steel grade	(mm)	Nominal ultimate load kN (100 200 400 1000 2000 5000)	Bell anchorage	QR plate anchorage	Square ribbed plate anchorage	Rectangular ribbed pl. anchorage	Square solid plate anchorage	Rectangular solid plate anchorage	LQ-plate anchorage	Ring plate anchorage	Multi area anchorage	Bond head anchorage	Loop anchorage	Coupler (movable)	Coupler (fixed)	Unbonded tendon
Smooth bar	1420/1570	10	●	●											●	●	
		12,2	●	●											●	●	●
	835/1230	26	●	●			●		●						●	●	●
		32	●	●		●	●	●	●						●	●	●
		36	●	●		●	●	●	●						●	●	●
	1080/1230	26	●	●	●		●		●						●	●	●
		32	●	●	●	●	●	●	●						●	●	●
		36	●	●	●	●	●		●						●	●	●
Thread bar	885/1080	15	●	●											●	●	●
	1325/1470	16	●	●			●								●	●	
	835/1030	26,5	●	●			●		●						●	●	●
		32	●	●		●	●	●	●						●	●	●
		36	●	●		●	●	●	●						●	●	●
	1080/1230	26,5	●	●	●		●		●						●	●	●
		32	●	●	●	●	●	●	●						●	●	●
		36	●	●	●	●	●		●						●	●	●
Strand	1570/1770	1x0,6"	●	●						●			●			●	●
		3	●	●						●			●	●		●	
		4	●	●						●			●	●		●	
		5	●	●						●			●	●	●	●	
		7	●	●							●		●	●	●	●	
		9	●	●							●		●	●	●	●	
		11	●	●						●	●		●	●	●	●	
		12	●	●						●	●	●	●	●	●	●	
		15	●	●							●	●	●	●	●	●	
		19	●	●							●	●	●	●	●	●	

DYWIDAG CABLES

Performance / Cable type	STRAND CABLE	BAR CABLE	
LOAD BEARING 1 CABLE (Static+Dynamic)	STRAND (7-wire stress-relieved) Grade St 1570/1770 Diameter 0,6" (Low relaxation)	THREADED BARS Grade St 835/1030 Dia. 32, 36 mm	STEEL TUBES Grade St 37 (DIN 17 100)
2 ANCHORAGE Static Dynamic Bending	 WEDGE ANCHORAGE Combined WEDGE+BOND ANCHOR STEEL TUBES	 NUT ANCHORAGE BOND ANCHORAGE STEEL TUBES	
CORROSION PROTECTION	Multiple anti-corrosion barriers to meet different requirements eg SHEATHING, GROUT, COATING OF STRANDS etc	eg STEEL TUBES, GROUT etc	
PRESTRESSING	SINGLE and MULTIPLE STRAND-JACKS	SINGLE BAR-JACKS	
CABLE ERECTION	Different procedures to fit into various construction demands eg Erection on site, Prefabrication		
ADDITIONAL FEATURES	FULLY REPLACABLE ADJUSTABLE before and after grouting	REPLACABLE ADJUSTABLE before grouting	


CABLE STAYS FOR BRIDGES
CABLE STAYS
EXAMPLES (STRUCTURE AND CONSTRUCTION METHOD)
PRESTRESSED CONCRETE
TYPICAL CROSS SECTION
CONCRETE GRADE B 45
160 / 135 CM
56 STRANDS TENDONS
19 ø 0,6"
STIRRUP
PROTECTIVE CONCR. LAYER WITH ADDIT. REINFORCEMENT IN VEHICLE IMPACT ZONE OF THE CONCRETE STAY
CONSTRUCTION ON SCAFFOLDING WITH POST-TENSIONED PRESTRESSING
STRUCTURAL CRITERIA
THE CONCR. STAY IS DESIGNED AS BEAM WITH FIXED ENDS STRESSED BY AXIAL TENSION FORCE AND STRESSED BY BENDING MOMENTS DUE TO DEAD WEIGHT OF STAY AND DUE TO WIND LOAD
CHECK OF STRESSES FOR PRESTR. CONCRETE MEMBERS ACC. TO GERMAN CODE DIN 4227
CHARACTERISTICS
-ALLOW. TENS. FORCE/SQM OF CONC. STAY-55 MN
-UNIFORM PRESTR. CONCR. DESIGN FOR TOTAL STRUCTURE INCL. STAY AND ANCHOR BLOCK
-STRUCTURE WITH SMALL DEFLECTIONS
-PERFECT PROTECTION AGAINST CORROSION
-RELIABLE RESISTANCE AGAINST VEHICLE IMPACT AND SIMILAR DAMAGES
-NO MAINTENANCE
-POSSIBILITIES FOR INDIVID. ARCHITECT. DESIGN
-LIMIT OF GIRDER SPAN AT 250 M APPROX. DUE TO DEAD WEIGHT OF CONCRETE STAY
PRESTRESSING STEEL
*COMPOSITE CROSS SECTION: WIRES ST. 1325/1470, DIA. 16 MM (W) + STEEL TUBE (ST)
*FIXED END: STEEL TUBE WITH INCREASED WALL THICKNESS
*LOAD TRANSFER AT ANCHORAGE: DEAD LOAD VIA THREADS OF WIRE (I) FATIGUE LOADS VIA BOND ANCHORAGE ZONE II
*COMPOSITE CROSS SECTION: BARS ST. 932/1079, DIA. 32 & 36 MM (B) + STEEL TUBE (ST)
*FIXED END: STEEL TUBE
*LOAD TRANSFER AT ANCHORAGE: DEAD LOAD VIA THREADS OF BARS (I) FATIGUE LOADS VIA BOND ANCHORAGE ZONE II
*COMPOSITE CROSS SECTION: STRANDS ST. 1580/1860, DIA. 0,6" (S)
*FIXED END: ANCHOR STEEL TUBE
*LOAD TRANSFER AT ANCHORAGE: DEAD LOAD VIA WEDGES OF STRANDS (I) FATIGUE LOADS VIA BOND ANCHORAGE ZONE II
DANUBE RIVER BRIDGE, METTEN, F.R.G.
ELEVATION
145,0
A-A
B-B
30,0
4,2
DETAIL C'
E-E
DETAIL D'
CONSTRUCTION STAGES
INCREMENTAL LAUNCHING
CASTING OF STAY ON SCAFFOLDING
FLÖSSER BRIDGE, FRANKFURT/M, F.R.G.
ELEVATION
106,5
A-A
2,2
B-B
C-C
20,4
5,3
FREE CANTILEVER WITH TEMPORARY STEEL CABLES
CASTING OF STAY ON SCAFFOLDING
ARRANGEM. OF FREE CANTILEVER SEGMENTS
DESIGN WITH CABLE STAYS ARRANGED BELOW GIRDER
ELEVATION
98,8
DETAIL A'
B-B
C-C
MAIN RIVER BRIDGE, HOECHST AG, F.R.G.
ELEVATION
148,2
A-A
B-B
30,9
C-C
D-D
2,6
ANCHORAGE
IN TOWER
IN GIRDER
TOP VIEW
SIMULTANEOUS SEGMENTAL CONSTRUCTION
PENANG BRIDGE, MALAYSIA
ELEVATION
107,5
225,0
107,5
A-A
25,3
ANCHORAGE
IN TOWER
IN GIRDER
TOP VIEW
SEGMENTAL CONSTRUCTION
FA ROUTE 63 OVER MISSISSIPPI RIVER QUINCY, ILLINOIS, U.S.A.
ELEVATION
134,1
274,3
134,1
A-A
ANCHORAGE
IN TOWER
IN GIRDER
CONSTR. STAGES

Rehabilitation of Post-Tensioned Bridge Deck

Klaus H. OSTENFELD
Head of Bridge Dep.
Cowiconsult

Virum, Denmark

Leif JONSEN
Chief Bridge Eng.
Cowiconsult

Virum, Denmark

The bridge was constructed in 1967 and hereafter regular routine inspections were carried out. In 1981 a major examination took place. Larger zones of deteriorated concrete were found in one of the cantilevered flanges, especially in areas near the abutments.

The bridge deck surface has a cross slope of 6% and consequently damage had only taken place in the lower cantilever, and fortunately not in the central part of the girder where the longitudinal main tendons are positioned. This distribution of the damage made repair possible. A full replacement of the superstructure would have resulted in an expenditure of more than 250% of the repair cost and almost total closure of one of the major motorways in Denmark.

The deterioration had started due to a leak in the waterproofing membrane. Unfortunately, the underneath concrete was not resistant against alkaline reactions.

- * - * - * - * - * -

After removal of the bridge deck pavement and water proofing, the deteriorated areas were surveyed by core drilling.

The deteriorated concrete was removed, however, in such a way that the reinforcement and prestressing tendons were not harmed. In this context it was very important that the transverse prestressing in the cantilevers could be reestablished. The transverse prestressing tendons consist of 12 dia. 5 mm wires. In order to protect these wires, the original sheath and grout were not removed until just before the new sheathing was ready to be installed. As it can be seen from the original transverse tendon arrangement, it was not possible to reuse the existing anchorages.

A new system was developed by Skandinavisk Spændbeton. A simple bar anchorage with thread and nut was connected to the wires. The details are shown on the poster. After casting the deck the wires were re-stressed and grouted.

Due to the repair work the longitudinal prestressing forces were lost in the cantilever. Consequently, it was necessary to place some additional ordinary reinforcement in the longitudinal direction, especially at the end of the bridge deck.

REHABILITATION OF POST-TENSIONED BRIDGE DECK

Bridge data

Year of construction: 1967
Year of repair : 1982

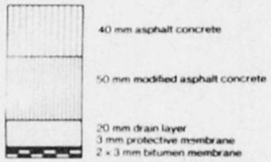
Post-tensioned concrete superstructure:
Ultimate cylinder strength, $\sigma_c \geq$ 45 MPa

Longitudinal prestressing:
12 × 0.5" strands, grade 160/180

Transverse prestressing:
12 × 5 mm wires, grade 150/170

System Freyssinet

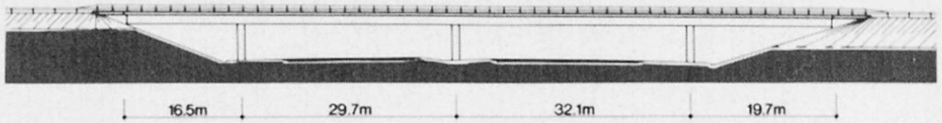
New waterproofing and pavement:



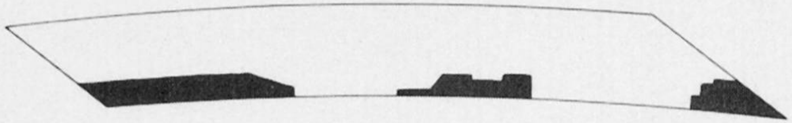
Bridge before repair

Deteriorated concrete has been removed

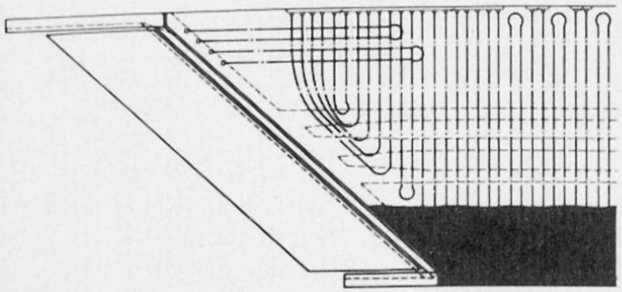
Motorway Bridge, Denmark



Plan

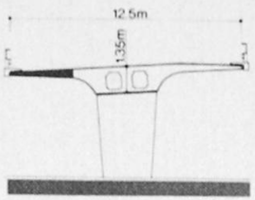


Plan section

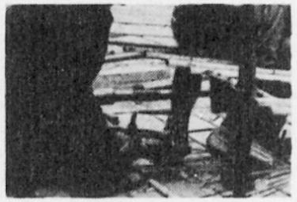


Existing transverse and longitudinal prestressing. The 12 × 5 mm transverse prestressing wires were re-used, but cut off at the edge of the bridge deck.

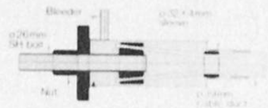
Cross section



Cantilever section

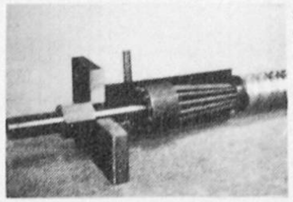


After cut-off, the 12 × 5 mm bars were prepared for coupling with simple bar anchorages. The 5 mm bars were provided with button heads on site.



Coupler and anchorage principle. (System developed by Skandinavisk Spændbeton).

Re-established transverse prestressing.



Section through coupler and anchorage.

Wind Structure Interaction on 235 m Tall RCC TV Tower in Delhi

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Bangalore University
Bangalore, India

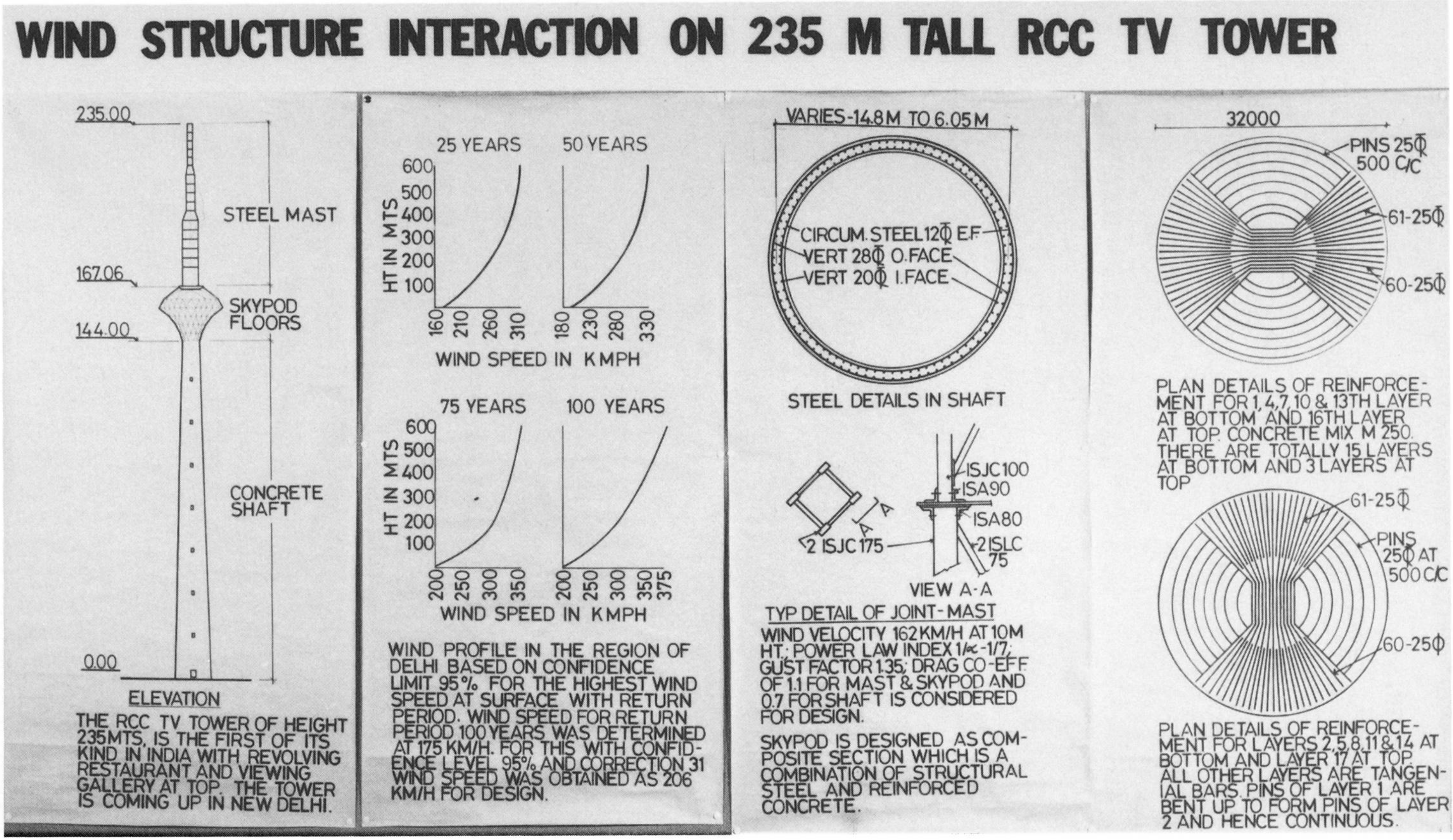
All India Radio has started the construction of an RCC TV tower, the first of its kind in India, 235 M height with revolving restaurant and viewing gallery at top, in the North - West suburban parts of Delhi, surrounded by industrial and residential area.

In connection with the above design Indian Meteorological Department (IMD) was requested to undertake analysis of surface and pilot baloon winds at Delhi air-port, to obtain wind escalation law. Utilising available surface upper wind and temperature data spread over long periods, IMD obtained the extreme values of wind and return period with different confidence limits. IMD further corrolated the IS code wind figures with the extreme values obtained by their study, indicating the probabilities of occurance of tornado and storms with return period, by determining maximum wind causing collapse of the structure.

The highest wind value recorded was 159 km/h in 1960. Wind speed for return period 100 years was determined at 175 km/h. For this, with confidence level (95%) and correction (31) wind speed was obtained as 206 km/h for the design. Since Delhi was affected by severetornado in 1978 with damaging wind speed of 250 km/h, the value of the power law co-efficient was worked out to be 1/9 under unstable and neutral conditions and 1/3 for stable conditions.

The power-law co-efficient 1/7 suitable for the change of gust speed with height was used for deriving wind profile.

Wind profile and a few structural details of pile cap, shaft and mast of this unique tower are shown in the photograph.


WIND STRUCTURE INTERACTION ON 235 M TALL RCC TV TOWER
235.00
STEEL MAST
167.06
SKYPOD FLOORS
144.00
CONCRETE SHAFT
0.00
ELEVATION
THE RCC TV TOWER OF HEIGHT 235MTS, IS THE FIRST OF ITS KIND IN INDIA WITH REVOLVING RESTAURANT AND VIEWING GALLERY AT TOP. THE TOWER IS COMING UP IN NEW DELHI.
25 YEARS
50 YEARS
HT IN MTS
600 500 400 300 200 100
160 210 260 310
180 230 280 330
WIND SPEED IN KMPH
75 YEARS
100 YEARS
200 250 300 350
200 250 300 350 375
WIND SPEED IN KMPH
WIND PROFILE IN THE REGION OF DELHI BASED ON CONFIDENCE LIMIT 95% FOR THE HIGHEST WIND SPEED AT SURFACE WITH RETURN PERIOD. WIND SPEED FOR RETURN PERIOD 100 YEARS WAS DETERMINED AT 175 KM/H. FOR THIS WITH CONFIDENCE LEVEL 95% AND CORRECTION 31 WIND SPEED WAS OBTAINED AS 206 KM/H FOR DESIGN.
VARIES-14.8M TO 6.05M
CIRCUM. STEEL 12Φ E.F.
VERT 28Φ O. FACE
VERT 20Φ I. FACE
STEEL DETAILS IN SHAFT
ISJC 100
ISA 90
ISA 80
2 ISLC 75
2 ISJC 175
A
A
VIEW A-A
TYP DETAIL OF JOINT-MAST
WIND VELOCITY 162 KM/H AT 10M HT.; POWER LAW INDEX 1/α-1/7; GUST FACTOR 1.35; DRAG CO-EFF OF 1.1 FOR MAST & SKYPOD AND 0.7 FOR SHAFT IS CONSIDERED FOR DESIGN.
SKYPOD IS DESIGNED AS COMPOSITE SECTION WHICH IS A COMBINATION OF STRUCTURAL STEEL AND REINFORCED CONCRETE.
32000
PINS 25Φ 500 C/C
61-25Φ
60-25Φ
PLAN DETAILS OF REINFORCEMENT FOR 1, 4, 7, 10 & 13TH LAYER AT BOTTOM AND 16TH LAYER AT TOP. CONCRETE MIX M 250. THERE ARE TOTALLY 15 LAYERS AT BOTTOM AND 3 LAYERS AT TOP
61-25Φ
PINS 25Φ AT 500 C/C
60-25Φ
PLAN DETAILS OF REINFORCEMENT FOR LAYERS 2, 5, 8, 11 & 14 AT BOTTOM AND LAYER 17 AT TOP. ALL OTHER LAYERS ARE TANGENTIAL BARS. PINS OF LAYER 1 ARE BENT UP TO FORM PINS OF LAYER 2 AND HENCE CONTINUOUS.

Continuous Reinforced Concrete Pavement on Bridges

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Civil Eng.
Public Works Ministry, Bridges Office
Brussels, Belgium

Continuous reinforced concrete pavement is widely used in Belgium since 1970. It consists of a 20 cm thick slab lying on an adequate fundation.

The longer the segments are, without transversal joints, the better. It is therefore very useful to pass over the under bridges and to avoid expansion joints.

The problem is important in Belgium because of the high density of the highway network and the short intervals between bridges.

Small frame bridges (10-20 m span) can be overpassed without problem.

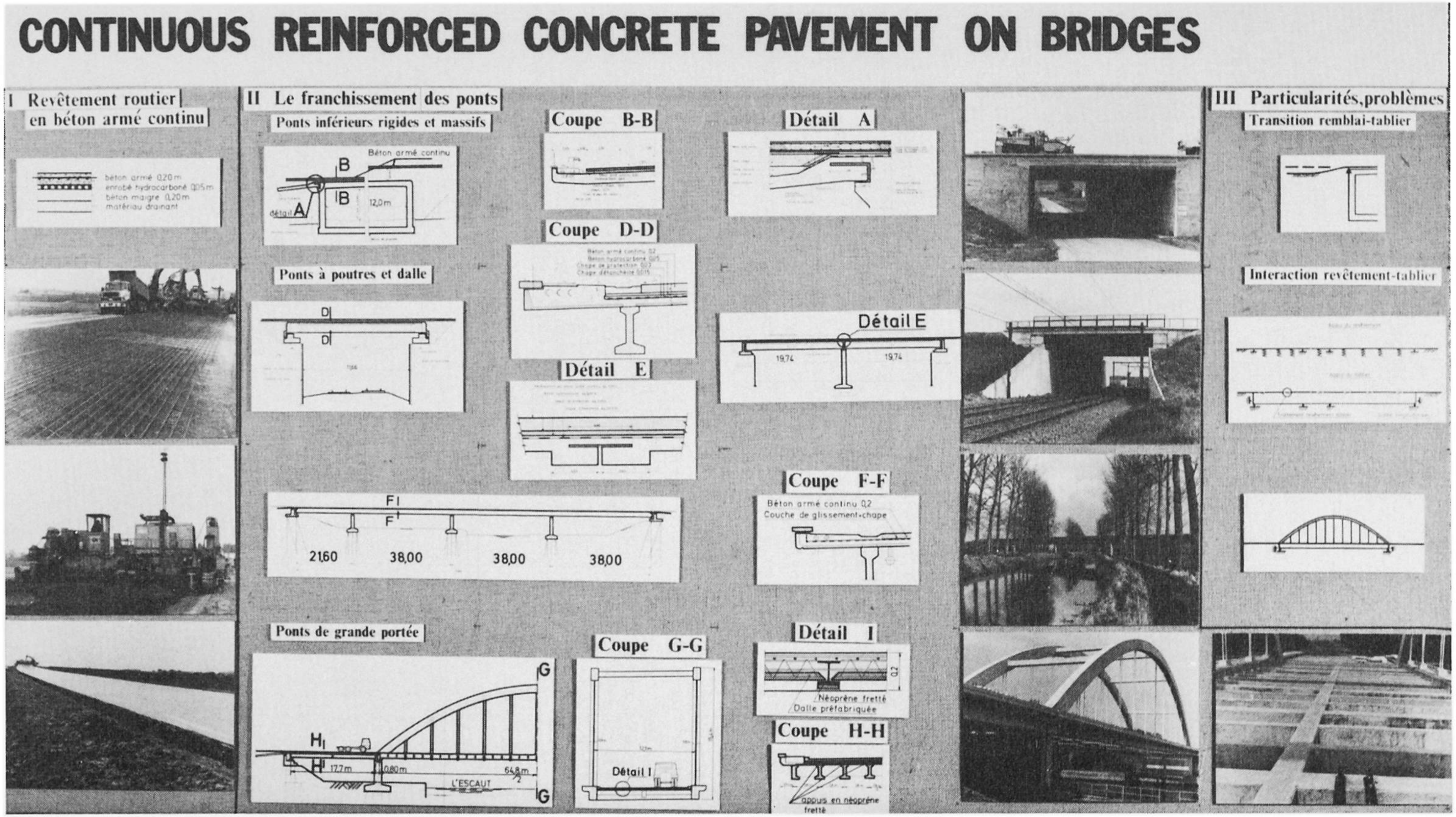
More complex is the case of bridges with precast guirders, the pavement lying on the deck slab with a friction interface.

The last evolution is to use the pavement as bridge slab, supported on the superstructure frame by "neopreen" bearings. Steel beams are incorporated in the pavement.

The main problems are :

- behaviour of the pavement between the embankments, behind the abutments, and the bridge (differential settling)
- interaction between pavement and bridge deck (deflection, expansion)

1400 Km continuous reinforced concrete pavement (2 ways) are in service in Belgium. The longest bridge overpassed is 136 m long and the greatest span is 65 m long.


CONTINUOUS REINFORCED CONCRETE PAVEMENT ON BRIDGES
I Revêtement routier en béton armé continu
béton armé 0,20 m
enrobé hydrocarboné 0,05 m
béton maigre 0,20 m
matériau drainant
II Le franchissement des ponts
Ponts inférieurs rigides et massifs
Béton armé continu
B
B
détail A
12,0m
Coupe B-B
Détail A
Ponts à poutres et dalle
D
D
Coupe D-D
Détail E
Détail E
19,74
19,74
F
F
21,60
38,00
38,00
38,00
Coupe F-F
Béton armé continu 0,2
Couche de glissement+chape
Ponts de grande portée
G
G
H
H
17,7m
0,80m
64,8m
L'ESCAUT
Coupe G-G
Détail I
Détail I
0,2
Néoprène fretté
Dalle préfabriquée
Coupe H-H
appuis en néoprène fretté
III Particularités, problèmes
Transition remblai-tablier
Interaction revêtement-tablier

Saudi Arabia — Bahrain Causeway

D.W. BILDERBEEK
Civil Eng.
Ballast Nedam Groep
Amstelveen, the Netherlands

K.B. SVENSSON
Civil Eng.
Ballast Nedam Groep
Amstelveen, the Netherlands

General

The Causeway incorporates 5 bridges and 7 embankments. The superstructure of the bridges consists of 2 separate boxgirders, each 12,3 m wide, prestressed in both longitudinal and transverse direction.
The bridges have a spanlength of 50 m and are constructed as a "Gerber" structure: alternatively cantilever girders of 66 m and drop in spans (suspende girders) of 34 m. Both cantilever and suspended girders have been prefabricated on shore and are placed in position as a complete unit.
The substructure consists of prefabricated hollow prestressed piles with an outer diameter of 3,5 m. Bridge no. 5 has been founded on caissons due to the presence of an aquifer. Bridge no. 3 incorporates the main navigation span. This navigation span has been constructed in accordance with the prefabricated segmental system.
Spanlengths: 80-150-80 m. The cast in situ piers rest on direct foundations.

Durability

Special measures have been taken because of the very aggressive environment:
- Blast furnace Portland cement (slag content 70-80%) has been used. This cement has a high sulphate resistance and a hight impermeability.
- A max Cl^-ion content of 0,1% by weight of cement has been prescribed for the concrete. As a result the sand, dredged from the sea, had to be washed extensively.
- Cover to the rebar for the piles 70 mm, for the superstructure 50 mm.
- The piles have been epoxy coated from -2,0 CD to +4,0 CD (splash zone).

Design criteria

In accordance with AASHTO, with some exceptions regarding the loading:
- lane loads : 10 kN/m' lane.
- trucks : 2 trucks, one of 600 kN and one of 300 kN.
- future pipeline : 10 kN/m', situated 4,5 m outside centre line of bridge
- earthquake : static load consisting of 6% of the permanent vertical load.
- ship collision : -56000 kN on each of the two mainspan piers.
  -28000 kN on each of the side piers of the mainspan.
  -varying load of 300-1000 kN for the piers adjacent to navigation spans.

Materials

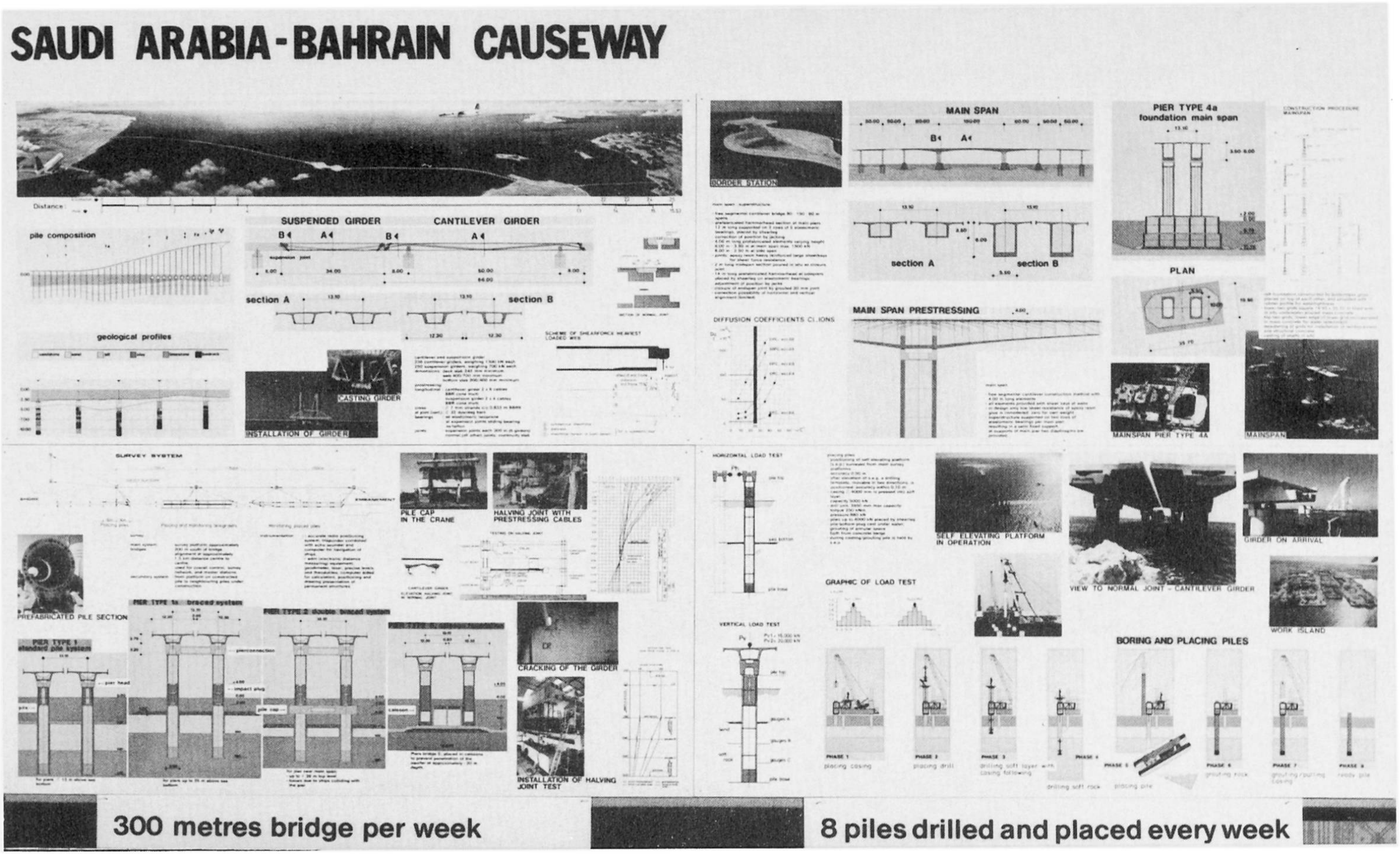
Concrete quality : f_c = 40 N/mm^2 for prefabricated part of halving joint.
f_c = 35 N/mm^2 for all structural elements.

Reinforcement steel : Fe B 400 and Fe B 500 (f_y = 400 N/mm^2 and 500 N/mm^2 respectively)

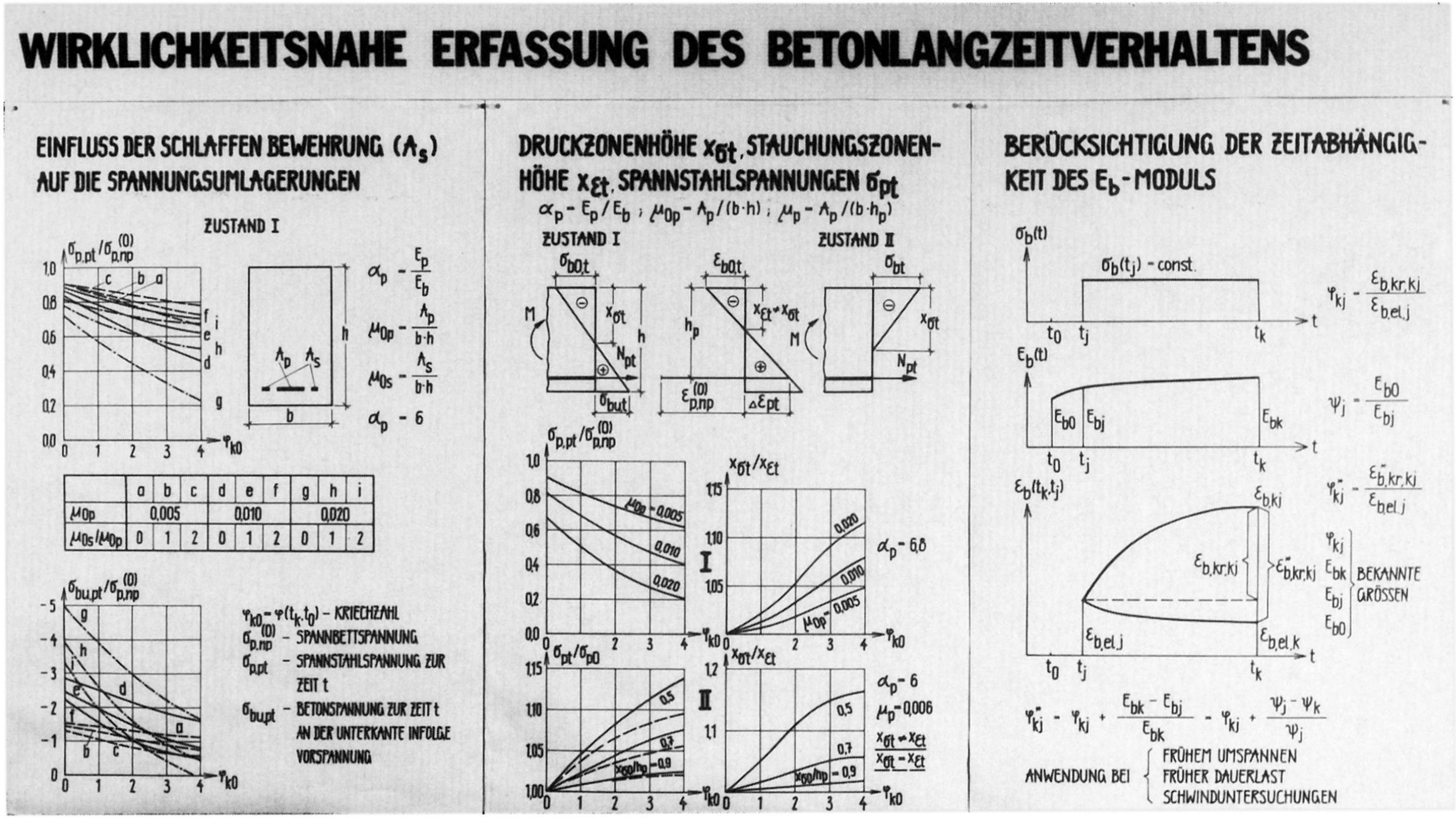
Prestressing steel : Longitudinal prestressing, consisting of BBR CONA-MULTI 0,62" strands: f_{pu} = 1770 N/mm^2.
Transverse prestressing, consisting of BBRV 7 mm wires: f_{pu} = 1670 N/mm^2.
Inclined Dywidag bars in halving joints f_{pu} = 1035 N/mm^2

Soil conditions

- sea bottom 4.00 to 12.00 minus CD.
- top layer of caprock 0.00 - 3.00 m thick.
- soft soil (sand/clay) 2.00 - 8.00 m thick.
- soft rock of claystone/siltstone.


SAUDI ARABIA-BAHRAIN CAUSEWAY
pile composition
SUSPENDED GIRDER
CANTILEVER GIRDER
section A
section B
geological profiles
CASTING GIRDER
INSTALLATION OF GIRDER
BORDER STATION
MAIN SPAN
section A
section B
MAIN SPAN PRESTRESSING
DIFFUSION COEFFICIENTS
PIER TYPE 4a
foundation main span
PLAN
MAINSPAN PIER TYPE 4A
MAINSPAN
SURVEY SYSTEM
PILE CAP
IN THE CRANE
HALVING JOINT WITH
PRESTRESSING CABLES
PREFABRICATED PILE SECTION
PIER TYPE 1
standard pile system
PIER TYPE 1a braced system
PIER TYPE 2 double braced system
CRACKING OF THE GIRDER
INSTALLATION OF HALVING
JOINT TEST
HORIZONTAL LOAD TEST
GRAPHIC OF LOAD TEST
VERTICAL LOAD TEST
SELF ELEVATING PLATFORM
IN OPERATION
VIEW TO NORMAL JOINT - CANTILEVER GIRDER
GIRDER ON ARRIVAL
WORK ISLAND
BORING AND PLACING PILES
PHASE 1
PHASE 2
PHASE 3
PHASE 4
PHASE 5
PHASE 6
PHASE 7
PHASE 8
300 metres bridge per week
8 piles drilled and placed every week

Wolfgang KRUGER
Prof. Dr., TU Dresden, Dresden, DDR



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ANALYSIS OF CONCRETE PIER WITH ASEISMATIC WALL

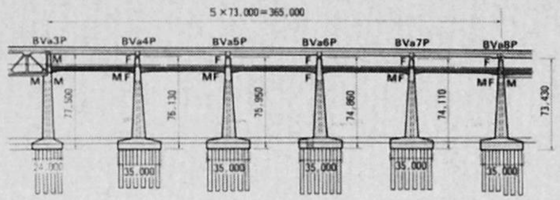


Fig-1 General view of Bannosu Viaduct

1. Bannosu viaduct, the highway-railway combined bridge, has high-rise piers with piles of 3 m in diameter.

Elasto-plastic FEM analysis and loading experiment with a 1/10 model were carried out to evaluate and to estimate the ultimate bearing capacity of concrete pier with aseismatic wall as well as the horizontal displacement at the rail level.
The cases of analysis by FEM consist of fourcases as shown in Table-1
The model for analysis is two dimensional and composed of concrete elements, reinforced ones, and bond ones to link both.

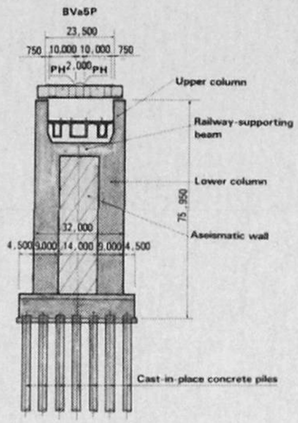


Fig-2 Pier (No.5)

2. Consideration

As shown in Fig-8, the pier does not show sudden deformation and can keep its shape at the time of cracks generation or reinforcements yielding.
This is also confirmed by the experiment with a 1/10 model.
The ultimate bearing capacity of the lower column is judged to be more than 3.5 times the design load.

Table-1 Analysis case

Case No.	Tensile strength of concrete	Ratio of reinforcements (%)	Purpose of analysis
Case-1	Considered up to σ_t=28.5kgf/cm² for both column and wall	0.5	Basic case.
Case-2	Considered up to σ_t=28.5kgf/cm² for column only	0.5	Influence of initial deterioration of wall due to shrinkage etc.
Case-3	Ignored for both.	0.5	Influence of initial deterioration of pier due to earthquake along bridge
Case-4	Ignored for both.	0.3	Influence of ratio of reinforcements as well as case-3

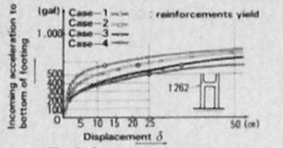


Fig-8 Ground acceleration vs displacement (at 1262nd nodal point)

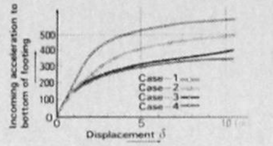
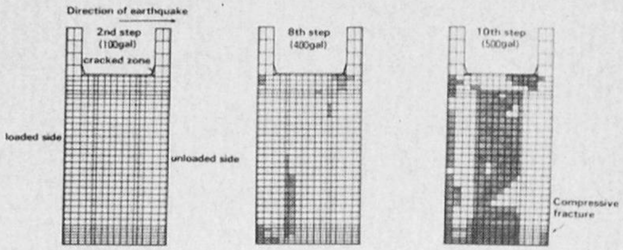


Fig-9 Close-look-up ground acceleration vs displacement (at 1262nd nodal point)

Followings show progress of concrete cracking (Case-1)



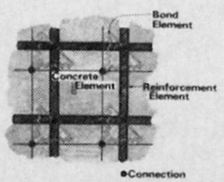


Fig-3 FEM Element

• Concrete property
Young's modulus
1st gradient $E_c=2.85\times10^5 t/m^2$
2nd gradient $E_{c2}=2.13\times10^5 t/m^2$
3rd gradient $E_{c3}=0.0$
Poisson's ratio
$\nu=0.1667$

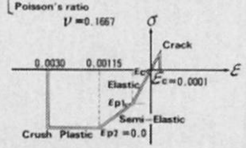


Fig-4 Concrete stress vs strain

• Reinforcements property
Young's modulus
1st gradient $E_s=2.1\times10^7 t/m^2$
2nd gradient $E_{sp}=2.1\times10^4 t/m^2$

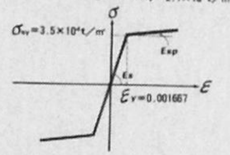


Fig-5 Reinforcements stress vs strain

• Bond property
Bond stiffness
1st gradient $G_s=1.0\times10^7 t/m^3$
2nd gradient $G_{sr}=1.8\times10^6 t/m^3$

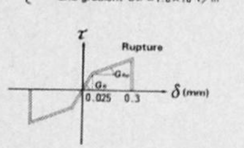


Fig-6 Bond stress vs displacement

Distribution of acceleration

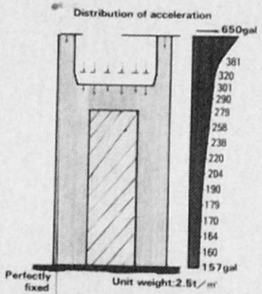


Fig-7 Distribution of load

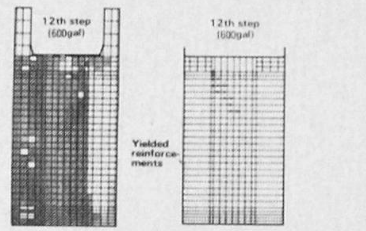


Fig-10 Progress of destruction (Case-1)

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