

Posters

Objektyp: **Group**

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

Band (Jahr): **11 (1980)**

PDF erstellt am: **16.07.2024**

Nutzungsbedingungen

Die ETH-Bibliothek ist Anbieterin der digitalisierten Zeitschriften. Sie besitzt keine Urheberrechte an den Inhalten der Zeitschriften. Die Rechte liegen in der Regel bei den Herausgebern.

Die auf der Plattform e-periodica veröffentlichten Dokumente stehen für nicht-kommerzielle Zwecke in Lehre und Forschung sowie für die private Nutzung frei zur Verfügung. Einzelne Dateien oder Ausdrucke aus diesem Angebot können zusammen mit diesen Nutzungsbedingungen und den korrekten Herkunftsbezeichnungen weitergegeben werden.

Das Veröffentlichen von Bildern in Print- und Online-Publikationen ist nur mit vorheriger Genehmigung der Rechteinhaber erlaubt. Die systematische Speicherung von Teilen des elektronischen Angebots auf anderen Servern bedarf ebenfalls des schriftlichen Einverständnisses der Rechteinhaber.

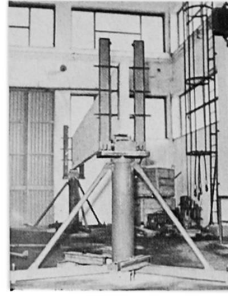
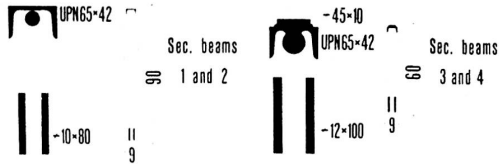
Haftungsausschluss

Alle Angaben erfolgen ohne Gewähr für Vollständigkeit oder Richtigkeit. Es wird keine Haftung übernommen für Schäden durch die Verwendung von Informationen aus diesem Online-Angebot oder durch das Fehlen von Informationen. Dies gilt auch für Inhalte Dritter, die über dieses Angebot zugänglich sind.

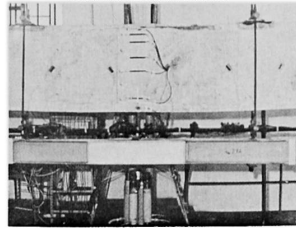
UNIVERSITA' degli
STUDI di BARI
FACOLTA' di INGEGNERIA
ISTITUTO
di
SCIENZA delle COSTRUZIONI
viale Japigia, 182 Bari (Italy)

TRIALS ON BEAMS IN METAL TRESTLE BURIED IN CONCRETE

G. DONATONE - G. FRADDOSIO - A. SOLLAZZO

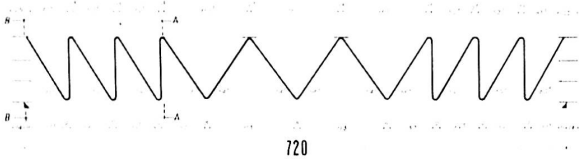


BEAM WITH BEARING DEVICES

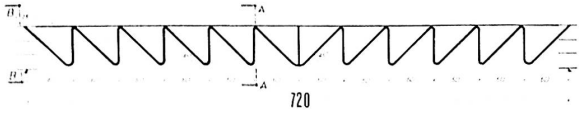


BEAMS AFTER FAILURE

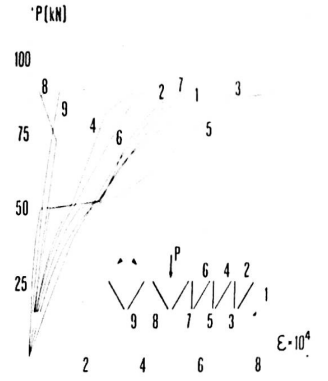
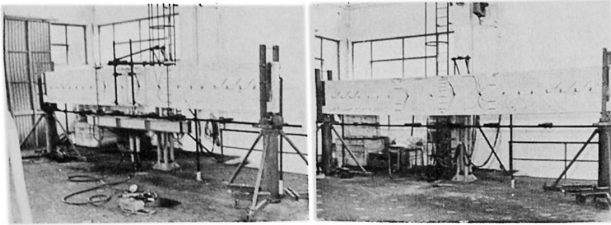
TRESTLE OF THE BEAMS 1-2



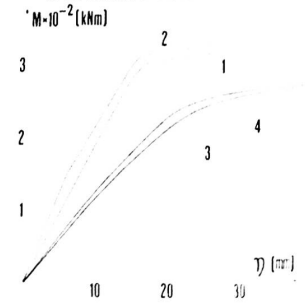
TRESTLE OF THE BEAMS 3-4



LOAD DEVICES AND BEAM AFTER CRACKING



STRAINS IN VERTICAL RODS AND DIAGONALS OF THE TRESTLE OF THE BEAM 1



DIAG. MOMENTS - DEFLECTIONS FOR THE BEAMS 1 2 3 4

TRIALS ON BEAMS IN METAL TRESTLE BURIED IN CONCRETE

Giovanni DONATONE - Giuseppe FRADDOSIO - Alfredo SOLLAZZO

Istituto di Scienza delle Costruzioni - Facoltà d'Ingegneria

Università degli Studi di Bari - (Italy)

SUMMARY

The research aims to investigate in theoretical and experimental way the behaviour of beams, with rectangular cross section, made by a welded metal trestle buried in concrete.

Have been tested four beams on a span of 7,20 m; the first two (n.1 and n.2) have a cross section of 9x90 cm while the other two (n.3 and n.4) have a cross section of 9x90 cm. Such beams must be considered as the webs of structural elements to complete during the installation by means of an upper slab in such way to give them a T section. They are to be used for a particular prefabrication system of multistoried buildings in which beam and partition are made by an only prefabricated block.

In the poster are shown the construction details of the prototypes, the load and bearing devices and the beams after failure.

Special "diapason" bearings have been designed to prevent only the beam rotation around its longitudinal axis and loads have been applied by means of previously calibrated hydraulic jacks.

Experimental results obtained point out that the considered beams have a behaviour very near to that of reinforced concrete beams, both under exercise loads and up to the rupture. In fact, as it is possible to see from the diagram shown in the poster for the beam 1, not only the diagonals near bearings, but also the vertical rods have resulted stretched; besides stresses in the former have always been higher than in the latter, as commonly happens for bended bars and stirrups. Rupture experimental moments, besides, are near enough to the theoretical ones valued by means of limit design theory for reinforced concrete beams, with deviations respectively of 1,5% and 7,5 for the beams n.1 and n.2 and of 5% for the beams n. 3 and n.4. Also compression strains in concrete and steel have been near enough to the theoretical ones. Failure announced by the appearance of many cracks, manifested itself through a sudden lateral buckling of structures under loads lightly higher than those for which strains in stretched steel, corresponding to yield point, had been measured. Thus it is to think that collapse happened just for reaching, in center line, of theoretical crisis situation and that only consequently, because of beams slenderness, lateral buckling occurred with contemporary instability of compressed stringer.

REFERENCES

- 1 - G. DONATONE - G. FRADDOSIO - A. SOLLAZZO: Risultati di esperienze su prototipi di travi a traliccio metallico immerso nel conglomerato cementizio. Atti dell' Istituto di Scienza delle Costruzioni dell'Università di Bari, n. 126; 1979.
- 2 - G. DONATONE - G. FRADDOSIO - N. SCATTARELLI: In tema di sperimentazione su travi a traliccio metallico immerso nel conglomerato cementizio. Atti dell'Istituto di Scienza delle Costruzioni dell'Università di Bari, n. 130, 1980

PRESTRESSED SLABS-DEVELOPMENTS IN EUROPE

P. Schüb
LOSINGER LTD
 Berne - Switzerland

RESEARCH

Development of prestressed concrete slabs
 615 x 20 x 40 cm test on 2 thin layers
 615 x 20 x 40 cm test on 2 thin layers
 Institute of Technology (TU), Berlin, 1977

615 x 20 cm PLATE TESTING WITH PLATE STRIPS

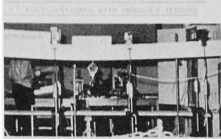


Fig. P5 4 - Test arrangement

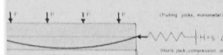


Fig. P5 5 - Test arrangement for plate strips

Specimen	Span (cm)	Area (cm²)	Yield strength (N/mm²)	Modulus of elasticity (N/mm²)	Deflection at failure (mm)	Failure load (kN)	Failure load (kN/cm²)
1	300	1000	450	25000	1.5	25.0	2.5
2	300	1000	450	25000	1.5	25.0	2.5
3	300	1000	450	25000	1.5	25.0	2.5
4	300	1000	450	25000	1.5	25.0	2.5
5	300	1000	450	25000	1.5	25.0	2.5
6	300	1000	450	25000	1.5	25.0	2.5
7	300	1000	450	25000	1.5	25.0	2.5
8	300	1000	450	25000	1.5	25.0	2.5
9	300	1000	450	25000	1.5	25.0	2.5
10	300	1000	450	25000	1.5	25.0	2.5

Fig. P5 6 - Characteristics of test specimens



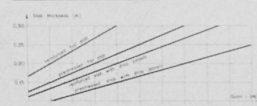
Fig. P5 7 - Results: Load-deflection curves for all plate strips

DESIGN

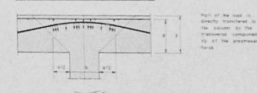
Scheme of load transfer by tendons



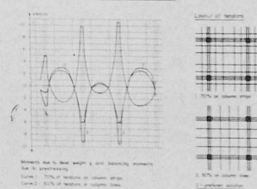
Slenderness of slabs



Punching mechanism

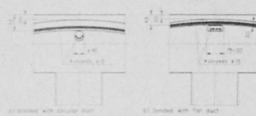


Distribution of tendons

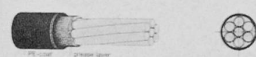


CONSTRUCTION

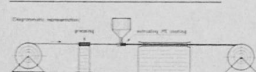
Excentricities



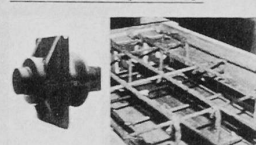
Unbonded monostrand



Extruding of unbonded monostrands



Monostrand stressing anchorage



EXAMPLES OF APPLICATION

Multi-Storey Car Park, Saas-Fee, Switzerland

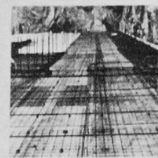
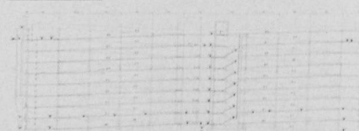
POST-TENSIONING WITH UNBONDED MONOSTRANDS

OWNER: Municipality of Saas-Fee
 ENGINEER: Schmidler und Partner AG
 CONTRACTOR: Atzenberger & Partner AG
 PRESTRESSING: Spanastone AG Lyssach

Structure with 5 prestressed floors, 62.2 x 34.8 m each, for parking of 300 cars

Span: 30.17 m
 Thickness: 0.20 m
 Loadings: dead load: 2.0 kN/m²
 live load: 2.5 kN/m²
 Prestressing steel: 2 J 1400

LONGITUDINAL SECTION



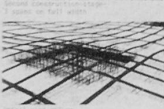
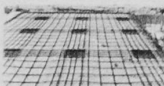
Underground Garage, Housing Complex Oed XII, Linz, Austria

POST-TENSIONING WITH BONDED TENDONS IN SLAB (SCL)

OWNER: Stadt Linz
 ENGINEER: Stöckl, Sigl & Partner AG
 CONTRACTOR: Oed XII 14 Baug AG
 PRESTRESSING: Spanastone AG Lyssach

Underground garage with a total area of 2000 m² x 100 x 40 m

Span: 7.90 m
 Thickness of slab: 0.20 m
 Loadings: dead load: 2.0 kN/m²
 live load: 2.5 kN/m²
 Prestressing steel: 2 J 1400





PRESTRESSED SLABS DEVELOPMENTS IN EUROPE

Peter Schlub
 Project Engineer
 Losinger Ltd.,
 Berne, Switzerland

The development of prestressed slabs in Europe was delayed in comparison with the USA and Australia.

Main reason for that delay was the missing of suitable standards and simplified design methods. With the research done (specially in Germany and Switzerland), standards and design methods could be established.

Today, recommendations are available in the United Kingdom (1) and have also been published by FIP (2). In Germany (3), Switzerland (4) and the Netherlands these standards are under preparation and will be issued shortly.

Most of the questions during the poster-session at the congress did concerne bonded versus unbonded solution, e.g. protection against corrosion, fire and earthquake behaviour.

Following the advantages respectively of unbonded and bonded systems.

Unbonded

- Maximum possible tendon drape
- No grouting required
- Corrosion protection of tendons also during transport, handling and placing
- Simple and fast placing of tendons
- Small friction losses
- Considerable dissipation of energy

Bonded

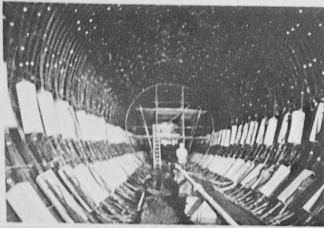
- Increased ultimate moment
- Local failures of tendons have only localised effects (e.g. in the case of fire, explosion and earthquake)

Finally, a summary of advantages of prestressed slabs:

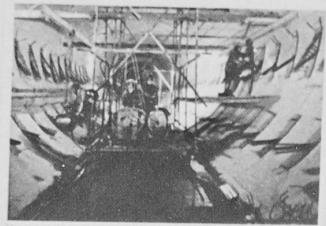
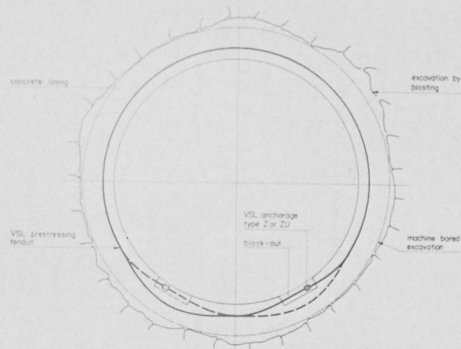
- . Economical
- . Increased span lengths and span/depth ratios
- . Reduced dead weights and building heights
- . Deflection and crack free under permanent loading
- . Improved punching shear resistance
- . Reduced construction time due to early stripping

References:

1. Flat slabs in post-tensioned concrete with particular regard to the use of unbonded tendons—design recommendations.
 Concrete Society Technical report No. 17, published 1979 by C & CA, Wexham Springs, Slough SL3 6PL.
2. Recommendations for the design of flat slabs in post-tensioned concrete (using unbonded and bonded tendons), FIP/2/5, May 1980, published by C & CA, Wexham Springs, Slough SL3 6PL.
3. DIN 4227, Teil 6 "Bauteile mit Vorspannung ohne Verbund"
4. SIA 162, Arbeitsgruppe 5, "Bruchverhalten von Platten"

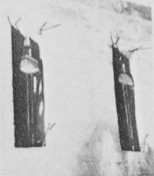


Photograph of the tunnel under construction, Berne, Switzerland, 1970-1972. Photo: Flughafen-Unternehmung AG, Berne.



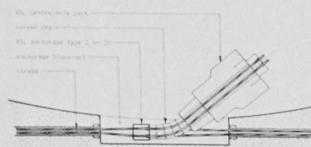
Photograph of the tunnel under construction, Berne, Switzerland, 1970-1972. Photo: Flughafen-Unternehmung AG, Berne.

Stressing Anchorage

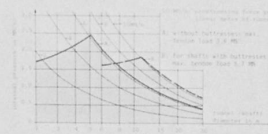


• VSL stressing anchorage type Z and ZU (ZU: ZU) • VSL stressing tendon • VSL prestressing tendon

Stressing Principle



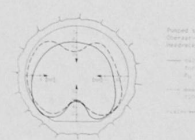
Range of Application



Prestressed Tunnel and Shaft Connections



Calculated / Measured Deformations



Representative Projects

PRESSURE TUNNELS

PIASTRA-ADDONDI, ITALY 1971/74	1200 m	1.2 m	1.2 m
Max. internal pressure	1.2 MPa	1.2 MPa	1.2 MPa
TALINO, SARONNO, ITALY 1971/74	400 m	1.2 m	1.2 m
Max. internal pressure	1.2 MPa	1.2 MPa	1.2 MPa
DEBAR-CHIMEL, SWITZERLAND 1971	100 m	1.2 m	1.2 m
Max. internal pressure	1.2 MPa	1.2 MPa	1.2 MPa
CHISTAS-PIASTRA, ITALY 1971/74	400 m	1.2 m	1.2 m
Max. internal pressure	1.2 MPa	1.2 MPa	1.2 MPa

SURGE SHAFTS

BRASINONE, ITALY 1971/74	100 m	1.2 m	1.2 m
Max. internal pressure	1.2 MPa	1.2 MPa	1.2 MPa
TALINO, SARONNO, ITALY 1971/74	400 m	1.2 m	1.2 m
Max. internal pressure	1.2 MPa	1.2 MPa	1.2 MPa
CHISTAS-PIASTRA, ITALY 1971/74	400 m	1.2 m	1.2 m
Max. internal pressure	1.2 MPa	1.2 MPa	1.2 MPa



PRESTRESSED PRESSURE TUNNELS AND SHAFTS

Igor Uherkovich, Francis Fink
LOSINGER LTD., VSL International

Where in tunnels and shafts the lack of sufficient overburden does not permit the rock to accept the internal pressure, or where this pressure is so high that the watertightness is in doubt although the stability of the tunnel shell is not in question, the structure is usually provided with a steel lining. Very often, however, transportation to remote sites as well as difficult installation condition make such a lining very expensive. The idea was to use the already existing concrete backfill as an autonomous lining without the need of a steel shell. This is possible with the help of the prestressing technique, using annular tendons acting like barrel hoops. To avoid the need of buttresses to anchor the tendons a special "floating" type of anchorage and the relevant stressing equipment as shown on the opposite page have been developed.

Many problems in the structural design and the construction had to be solved since in view of the often unpredictable behaviour and embedment the design and construction of underground constructions cannot entirely be carried out on the basis of the principles applied for open-air structures. Prestressed tunnel linings subject to high water pressures require a special treatment of the contact surface between rock and concrete. After pressing the resulting gap between rock and concrete has to be filled using the traditional grouting techniques. Also important is the use of a suitable formwork construction to ensure a complete concrete filling.

The proposed solution is not only limited to straight cylindrical sections of tunnels and shafts but can also be applied economically for tunnel and shaft connections, by-passes, etc.

A number of prestressed pressure shaft and surge chamber projects have been carried out successfully using this method. Noticeable reductions in construction time and cost savings were achieved. Although all completed projects were done in highly developed countries, still further advantages can be expected by using this solution in developing countries.



This project is a plan for constructing residential buildings higher than 14 stories with a total of some 3,400 residential units by different owners on the reclaimed land off the coast (see, Fig. 1). They are 14, 19, 24 and 29 stories and the variations of 11 in the type (see, Fig. 2). Fig. 3 shows an example of the plans of the residential units.

- * Chief Structural Engineer, Dr. Eng., Division of Structural Design, Takenaka Komuten Co., Ltd.
- ** Chief Structural Engineer, Division of Structural Design, Nippon Steel Corporation
- *** Chief Structural Engineer, Division of Structural Design, Takenaka Komuten Co., Ltd.

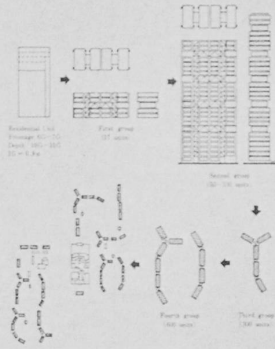


Fig. 1 System of High-Rise Residential Buildings



Fig. 2 Composition of Stories

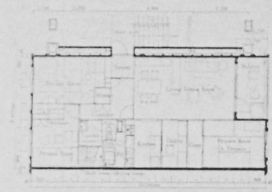


Fig. 3 Plan of A Residential Unit

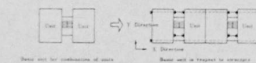


Fig. 4 Structural Framework

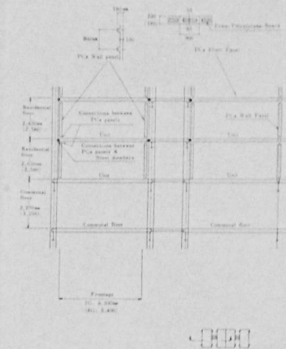


Fig. 5 Structure of Residential Units

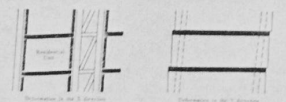


Fig. 6 Deformation of Residential Units



A Unique System of High-Rise Residential Buildings by Large Steel Structural Framework

T. HISATOKU, Dr. Eng. R. TAMURA Y. KATO
Chief Structural Engineer, Chief Structural Engineer, Chief Structural Engineer,
Takenaka Komuten Co., Ltd. Nippon Steel Corporation. Takenaka Komuten Co., Ltd.
Osaka, Japan Tokyo, Japan Osaka, Japan

1. THE OUTLINE OF PROJECT

This project (about 3,400 Residential units in 52 buildings) was completed in July, 1979. The name "ASTM" is the combination of the first letters of Ashiyahama, (name of the city where these buildings were built) and the five participating companies in Japan.

The plan submitted by the ASTM won the first prize for its unique system utilizing prefabrication and industrialization in August 1973 in the competition for High-Rise Housing Complex at Ashiyahama.

This project is a plan for constructing residential buildings higher than 14 stories with a total of some 3,400 residential units by different owners on the reclaimed land off the coast (see, Fig. 1). They are 14, 19, 24 and 29 stories and the variations of 11 in the type (see, Fig. 2). Fig. 3 shows an example of the plans of the residential units.

2. THE OUTLINE OF THE STRUCTURAL DESIGN

2.1 The Structural Frame

The structural frame of the residential buildings are shown in Fig. 4. The basic unit concerning the structure is four residential units per floor as shown in the figure. In the X direction (see Fig. 4) in order to create the free space for residential units, structural frame consist of two large rigid frames making the core with the stair column and the communal floors beams. In the Y direction (see Fig. 4), structural frame consists of four rigid joint truss frames situated at the both sides of the stairs.

2.2 The Structure of the Residential Unit

Fig. 5 shows the outline of the structure of the residential unit. The residential unit is composed of PCa panels (that is precast concrete panels), and the four stories residential units lie on the beam which is located on the upper floor of the communal floor, except the lowest part of the building. The PCa panels and the PCa wall panels bear the vertical load, and the load is transmitted from the PCa floor panels to the PCa wall panels, and the vertical load of the four stories is eventually supported by the beam of the upper floor of the communal floor. These PCa panels are not participated against wind or earthquake.

2.3 The Relationship between the Residential Unit and the Structural Framework

The walls and the floors of the residential unit are not only required to bear the vertical load but also to comply with deformation of the structural framework when horizontal loads are exerted on the structural framework. Taking these requirements into consideration, the design has been made about each of the directions as shown by Fig. 6. For this purpose, tetrafluoroethylene resins are placed on top of the walls of every story to slide bearing materials.

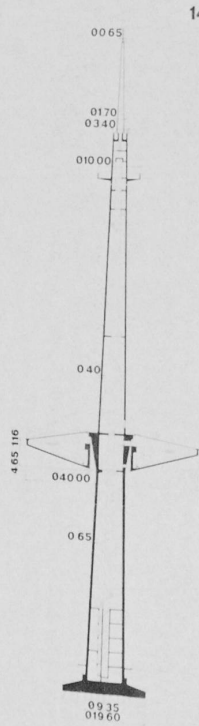
CHATEAU D'EAU ET MAT D'ANTENNES A MECHELEN

PROF. DR. IR. F. MORTELMANS

AVIS DE CONCOURS
 VILLE DE MECHELEN
 BUREAU DU PROJET
 PROF. DR. IR. F. MORTELMANS
 A. LEUVEN
 INGENIEURS CONGRES
 UNION TECHNIQUE DE
 LA CONSTRUCTION LTD
 BRUXELLES
 ENTREPRENEUR
 VAN NUYT VERBODER
 PRECONSTRANTE S.S.
 CARACTERE DU RESERVOIR: EXONERÉ
 COUT TOTAL DE LA CONSTRUCTION
 1000000 FR.
 DATE DE MISE EN SERVICE
 01.09.1978

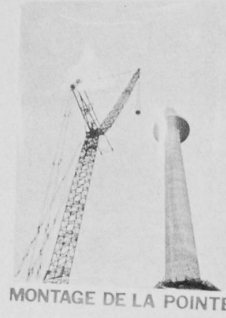


DETAILS DE LA PRECONTRAINTE DU RESERVOIR

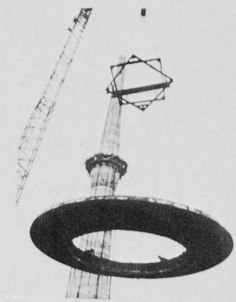


143.10
140.10

120.10
111.25



MONTAGE DE LA POINTE



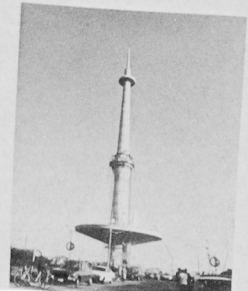
MONTAGE DE LA PLATE-FORME

50.00



FUT EN CONSTRUCTION

0.00
-6.20





CHÂTEAU D'EAU ET TOUR DE TELECOMMUNICATIONS
EDIFIE A MECHELEN EN BELGIQUE

Fernand MORTELMANS
Professeur Ordinaire
à la Katholieke Universiteit Leuven
Leuven-Heverlee, Belgique

La construction est composée :

- d'un réservoir d'une capacité de 2.500 m³ à un niveau d'eau maximum de + 50 m par rapport au sol,
- de l'emplacement de trois antennes paraboliques au niveau de la toiture du réservoir (+ 55 m),
- d'une plate-forme à + 110 m et sur laquelle sont montées les antennes réceptrices de la radio- et télédistribution de la ville,
- d'un mât en acier inoxydable de 20 m de hauteur et qui couronne la construction entière,
- d'un paratonnerre extensible de 4 m de hauteur au sommet du mât.

La gaine centrale en béton armé et de 120 m de hauteur fût réalisée par procédé de coffrage glissant. Le réservoir consiste d'un fond conique renforcé de 16 parois radiales en béton précontraint, une paroi intérieure, une paroi extérieure de 40 m de diamètre, également précontraintes. Le tout est couvert par une toiture en forme de coque mince conique reposant sur la paroi extérieure du réservoir. Après leur parachèvement sur le sol, le réservoir et la toiture ont été hissés vers une console de suspension et ancrés définitivement.

La plate-forme des antennes fût également construite sur le sol. Elle est composée de trois anneaux préfabriqués en béton léger, solidarisés par coulage d'une couche de béton après leur mise en place par une grue de 160 m de hauteur de levage. Le mât en acier inoxydable, mis en place par la même grue, n'a qu'une fonction purement esthétique.

Il était surtout la façon d'exécution de cet ouvrage d'art qui à suscité l'intérêt du public.

Apparemment les opinions sont unanimes sur le fait que les qualités esthétiques de cette construction peuvent être attribuées à l'élégance et la simplicité des lignes, le choix des matériaux et leur mise en oeuvre comme les combinaisons de béton lis et rugueux et l'acier inoxydable.

Finalement apparait la double dualité réservoir/plate-forme et gaine en béton/flêche en acier inoxydable.

Maître de l'ouvrage : La Ville de Mechelen
Auteur du projet : Prof.dr.ir. F. Mortelmans
Bureau d'Ingénieurs Conseils : I.T.H. Bruxelles
Système de précontrainte et de levage : V.S.L.
Pieux des fondations : Soc. Pieux Franck'i
Entrepreneur : Soc. Van Hout à Vosselaar (Belgique)
Coût des travaux : 80.000.000,- FB
Délai d'exécution des travaux : 200 jours ouvrables



Elementierter
Stahl-Hochbau
mit hohem
industriellen
Vorfertigungs-
grad

6D-Bauverfahren · Doubrava KG · Attmang · Austria

In- und
Auslands-
patente



BAUEN OHNE GERÜST

Die Entwicklung des 6D-Bauverfahrens stand unter dem Protektorat der Osterr. Forschungs-Förderung und gründet auf der exakten Auswertung der Erkenntnisse weltweiter Bauforschung

6D-Charakteristik:

- Gestaltungsvielfalt durch freie Addierbarkeit der selbsttragenden 6D-Raumeinheiten in allen 3 Dimensionen (bis 21 Etagen).
- Bleibende Flexibilität gegenüber beliebigen Raumgrößenveränderungen (keine tragenden Wände!).
- Hohe Wärme- und Kälteschutzwerte durch optimale bauphysikalische Detaillösungen. Alle geforderten Brandschutz-, Wärme- und Schall-Dämmwerte sind erfüllbar.
- Keine Gerüstung, keine Materialverluste auf der Baustelle.
- Kurze Bauzeit und daher Fixpreis.
- Erdbebensicher. Ideal exportfähig



150-BETTEN-HOTEL MIT RESTAURANT

TWO SPECIAL CHINESE TIMBER BRIDGES

TANG HUAN CHENG



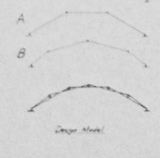
Rainbow Bridge

This is a Chinese national art treasure, the Longjiao "River Side School" (The Longjiao Pavilion). The bridge was constructed in year 1332, and was first repaired and enlarged by order in year 1953.



Dimensions of the bridge as determined from sketches of drawings by traditional methods in Longjiao, are shown on previous pages. Data taken by scientific construction, the timber beam segment is about 85 cm in diameter. The steel materials required is 13,422 kg for painting about 100 and steel plates.

Combined Beam-Arch Construction



The bridge structure consists of two basic systems, system A and B. Both systems are suitable construction, as they are well supported by the construction and timber bridge. The structure is designed as two bridge arch, but each system is built as a structural beam. It is named as "Combined Beam Arch Construction of timber bridge".



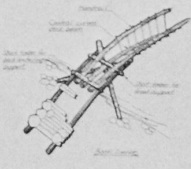
Bow Bow Bridge

In the North end of the construction structure, China, during the historical time period, there are some interesting timber bridge and structure by means of the wood. The construction and the structure, figure will show "Bow Bow Bridge".

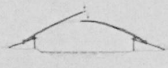
Load length 12-15 m



The bridge will make it constructed with three curved beams connected with 1" diameter steel members. The whole bridge made in a new method in the future view, and interesting shape in the case structure. Bow Bow bridge is a special structure.



The construction of the bridge is shown in the picture. Longjiao Bridge and structure in the structure on each side, and construction of the river. The gap between the construction of the construction of the construction. During the construction phase, the construction phase and along with timber bridge, the Bow Bow bridge is finally completed.



The Bow Bow bridge design in 2012 years, will be built under 20 is a professional structural design construction.

Conclusion

There are interesting special Chinese timber bridges are interestingly constructed. Historical in form, simple structural and design in detail. Their construction process will be used in the bridge design with new material and technology for new construction purposes.

prof. dr. kruno
TONKOVIĆ

4 STRUCTURES EN BOIS - WOOD - HOLZ - -

JUGOSLAVIJA

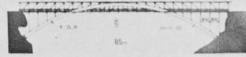
Zagreb - Jugoslavija
Inžinjerstvo
gradjevinski institut

IVBH
IABSE
AIPC

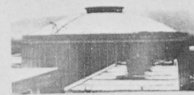
1980



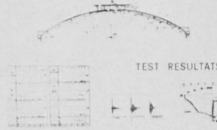
85m



S. Dimnik: Pasarela - Kokra - Kranj - Jugoslavija 1938



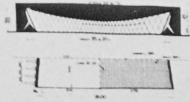
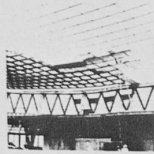
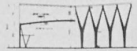
L = 39m



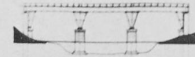
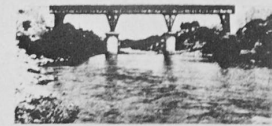
K. Tonković: Kupola brodarski institut - Zagreb - 1954

K. Tonković: hala velesajam - Zagreb

1955



L = 95m



K. Tonković: most Budak - Lika 1952

HARTL
HOLZKONSTRUKTIONEN
GESELLSCHAFT MBH

1190 WIEN, SEIFENRINGER STRASSE 2 - POSTFACH 46
TELEFON: 0227/32 25 55 - FERNSCHREIBER: 0744333
3754 WIRFELZ, NIEDERÖSTERREICH
TELEFON: 02766/237

