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

Band: 11 (1980)

Rubrik: IV. Special structures

WORKING SESSION

IV

Special Structures

Constructions spéciales

Spezielle Bauwerke

Co-chairmen: B. Højlund-Rasmussen, Denmark
N. Devres, Turkey

Coordinator: R. Favre, Switzerland

Leere Seite
Blank page
Page vide

IV

Two unusual Features of the Merivale Rail Bridge

Deux propriétés extraordinaires du pont de chemin de fer ,,Merivale''

Zwei ausserordentliche Eigenschaften der ,,Merivale'' Eisenbahnbrücke

JOHN SNELLING
Director
Cameron, McNamara & Partners Pty. Ltd.
North Brisbane, Queensland, Australia

SUMMARY
Two unusual features of the Merivale Rail Bridge across the Brisbane River in Australia are described; the cable pattern in the tied arch and its effect, and the foundations of the piers.

RESUME
Deux propriétés remarquables du pont de chemin de fer ,,Merivale'' traversant le fleuve ,,Brisbane'' en Australie sont décrites: le tracé des câbles dans l'arc et son effet, et les fondations des piles immergées.

ZUSAMMENFASSUNG
Zwei beachtenswerte Eigenschaften der ,,Merivale'' Eisenbahnbrücke werden hier beschrieben: der Kabelverlauf der Bogenbrücke und seine Wirkung sowie die Fundierung der Flusspfeiler.

1. INTRODUCTION

The Merivale Bridge across the Brisbane River, Australia carries a double track rail connection between the north and south side suburban rail systems in the City of Brisbane. The bridge has a number of unusual features, certainly unique in Australia, two of which are described herein.

2. MAIN SPAN

The main river navigation span is a 132m steel tied arch with sloping arches. Cross bracing between the arches has been eliminated after detailed buckling analysis. This system was chosen because

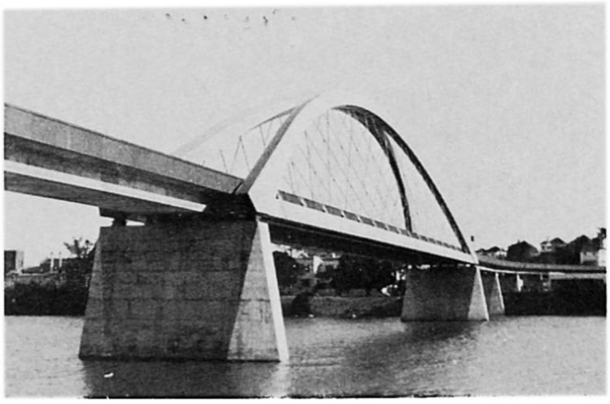


Fig.1. Merivale Bridge

1. Good appearance in the city environment was of the utmost importance.
2. Erection of the structure permitted minimum interruption to shipping.
3. Railway grading requirements and river navigation clearances were satisfied.
4. Stiffness under railway operations could be assured.
5. The structure could be relatively easily maintained.
6. Cost was acceptable.

With respect to item 4, an innovation in this bridge was the development of a cable arrangement in which the slope of the cables varies symmetrically about the centre of the arch. The effect is to inhibit distortions of the arches and ties, particularly in the outer quarter, under assymetric live loadings. The benefit can be illustrated by comparing the distortions under a partial load of similar structures with vertical, uniformly sloping and varying sloping cables. This is illustrated in Figure 2 where the deflection of arch and tie are shown much distorted. It can be seen that the uniformly sloping hanger pattern reduces the deflections by $4\frac{1}{2}$ times compared with the vertical pattern. The varying slope pattern reduces them by half again to be one ninth of the deflections of the span with vertical hangers. The theory has been verified in practice and the vibration characteristics of the span are remarkably good. Other beneficial effects are an estimated reduction of approximately 250 tonnes in the quantity of steel necessary for this bridge compared with one with constant slope hangers, and a reduction in the quantity of bolts needed for the splices in the arches. Moving locomotive tests have confirmed favourable vibration and stiffness characteristics.

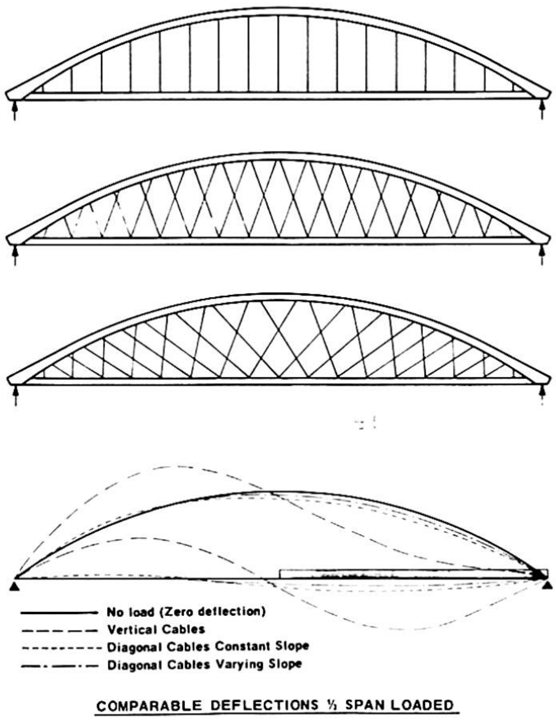


Fig.2

3. RIVER FOUNDATIONS

The river piers are founded on multiple 1220mm diameter steel cased cast insitu piles socketted into rock. To avoid the expense of cofferdams which have been troublesome elsewhere in this river, and precast concrete base skirting panels which would be vulnerable to damage by shipping at this location, and to minimise underwater work, a system was devised for the construction of the river piers whereby the bottom of the pier was formed as a pre-cast concrete boat, floated into position and sunk onto falsework. (Fig.3)

Holes had been formed in the bottoms of the boats for the piles to pass through

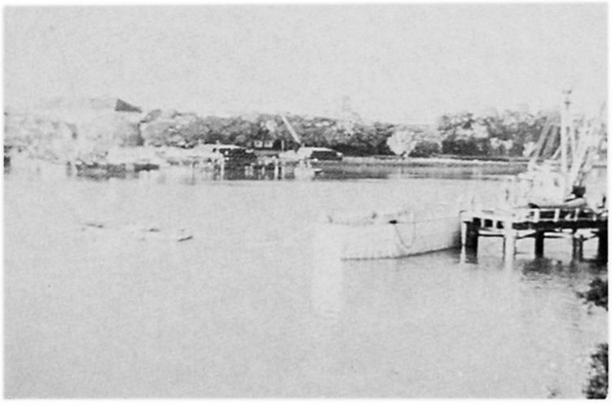


Fig.4
Pier base manoeuvring onto Falsework

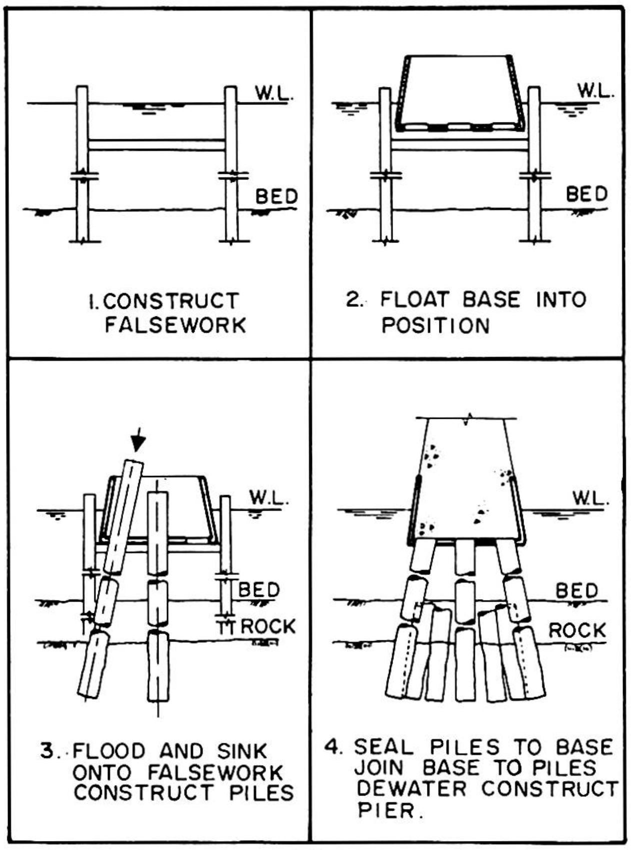


Fig.3
Construction Sequence

and temporary templates at the tops of boats fixed the directions of the piles which were driven to bedrock with diesel hammers.

On completion of the piling, the pile liners were fixed to the pier base by metal straps and the annular space between the pile liners and the base was sealed by a diver working in a metre of water - the only underwater work involved.

Once the seal was made, the base was dewatered and cleaned, and permanent brackets were welded between the pile liners and the base. Excess liners were cut off, the reinforcing steel placed, and the pier base cast.

The remainder of each pier was then completed conventionally.

Underwater work was avoided, the piles were accurately positioned and the system worked well with completely satisfactory results.

Leere Seite
Blank page
Page vide

IV

Curved Steel Guideway for Suspended Monorail System

Coulisse de guidage courbe en acier destinée à un système de monorail suspendu

Gekrümmte Stahl-Führungsschiene für ein schwebendes Einschienenbahn-System

T. YAMASAKI
General Manager
Res. Lab.
Kawasaki Steel Corp.
Chiba, Japan

T. YANO
Chief, City Planning Dep.
Chiba Pref. Government,
Chiba, Japan

Dr. H. HIKOSAKA
Assistant Professor
Kyushu University
Fukuoka, Japan

SUMMARY
A thin-walled steel beam with open cross-section, which is extremely weak in torsion, is used as the guideway for a rubber-tyred suspended monorail system. In order to understand clearly the behaviour of this special structure, both static and fatigue tests were carried out upon the half-scale model of an actual curved guideway beam.

RESUME
Une poutrelle d'acier à paroi mince et section transversale ouverte, de rigidité à la torsion extrêmement faible, sert de coulisse de guidage pour un système de monorail suspendu à pneu en caoutchouc. Des essais de charge et de fatigue ont été pratiqués sur un modèle à l'échelle 1/2 d'une coulisse de guidage courbe réelle dans le but de comprendre clairement le comportement de cette structure particulière.

ZUSAMMENFASSUNG
Ein dünnwandiger Stahlträger mit offenem Querschnitt, welcher extrem torsionsweich ist, wird als Führungsschiene für ein gummibandagiertes, schwebendes Einschienenbahn-System verwendet. Um das Betriebsverhalten dieser speziellen Struktur klar zu verstehen, wurden an einem Modell des gekrümmten Führungsschienenträgers (Massstab 1:2) Belastungs- und Dauertests durchgeführt.

1. STRUCTURAL DESCRIPTION

During the past several years increasing interest has developed in new forms of ground transportation which can provide automated high-capacity urban transportation. A number of advanced systems including personal rapid transit, dual mode, monorail and high speed ground system are under research and development in Japan. Chiba City, one of densely populated and highly industrialized cities in Tokyo metropolitan area, has recently decided to adopt a rubber-tired suspended monorail system as shown in Fig. 1. It is the first practical use of suspended monorail as a mass-transportation system in Japan. In the first stage of program it is planned to construct double track guideways of 17.5 kilometer length including twenty stations and terminals. One train is composed of four vehicles and the carrying capacity of this system becomes about 20 thousand passengers per hour. Since guideway cost usually far exceeds vehicle cost in this system, the technology of guideway design, construction and maintenance is extremely important. In Chiba monorail system thin-walled slender steel beams with large span are used as guideways, which can minimize aesthetic impact, required land area and construction cost. Silent and weather-proof operation is additional advantage in this system since runways, electric conductors and motor bogies are placed inside the guideway beams with box-type cross-section. We have, however, some unsolved technological problems about guideway design resulting mainly from existence of bottom split in the cross-section, which is necessary for passage of the pendulous suspenders connecting bogies to the vehicles.

Owing to this split the thin-walled guideway beams are of open cross-section, which is extremely weak in torsion and derives large warping stress. It influences the route-determination flexibility (permissible radius of curvature) because in the case of curved guideway beams large torsional moment arises from dead weight, vehicle loading, horizontal centrifugal force of running vehicle and wind force.

Typical cross-section of guideway in Chiba monorail is shown in Fig. 2. Only transverse stiffening frames of limited size are provided to prevent distortion

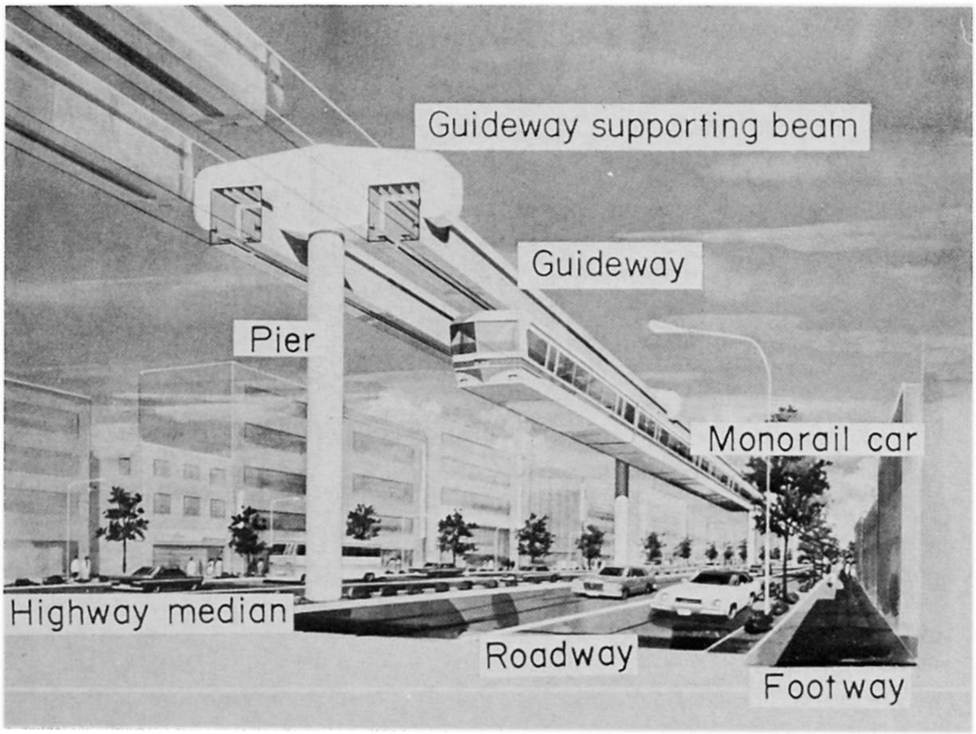


Fig. 1 Suspended monorail system

of the cross-section because it is impossible, in this special structure, to insert cross-bracings and lateral members for obtaining increased torsional stiffness.

Inevitable distortion of the cross-section will decrease the guideway stiffness and will induce additional stresses. The optimum size and spacing of the stiffening frame are therefore the most important factors considered in the guideway design. As the size and rigidity of runway beams are also limited, the local stress of runway beams beneath the wheel loads becomes another problem of consequence. In addition fatigue design of welded joints is very important in this structure since various initial imperfections can not be avoided in fabrication and serious local stresses are caused by repeated vehicle loads. In order to make clear the behavior of this special structure and to solve the above mentioned problems, both static and fatigue tests were carried out upon the half-scale model of an actual curved guideway beam.

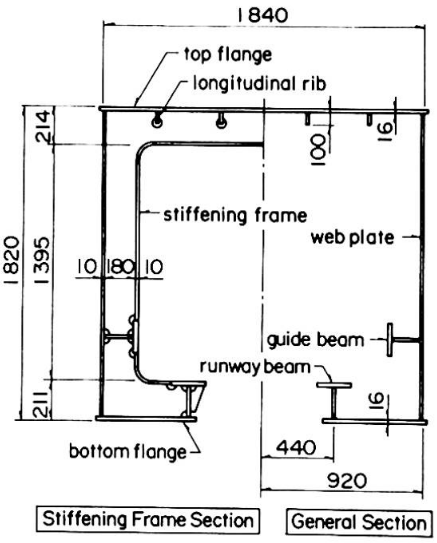


Fig. 2 Cross-section of guideway

2. EXPERIMENTAL MODEL, TESTING RIG AND INSTRUMENTATION

Dimensions of the simple span curved guideway model, which was used for both static and fatigue tests, are shown in Fig. 3. The model was fabricated from 9 mm thick steel plate (JIS-SS41) and transverse stiffening frames were positioned at regular intervals of 725 mm.

The static test was conducted under three different loading conditions, i.e., a vertical load divided into half point loads on the runway beams, a horizontal centrifugal load on outer guide beam and a horizontal centripetal load on inner guide beam. The maximum load applied in each test was chosen so that the stress of model did not exceed elastic limit.

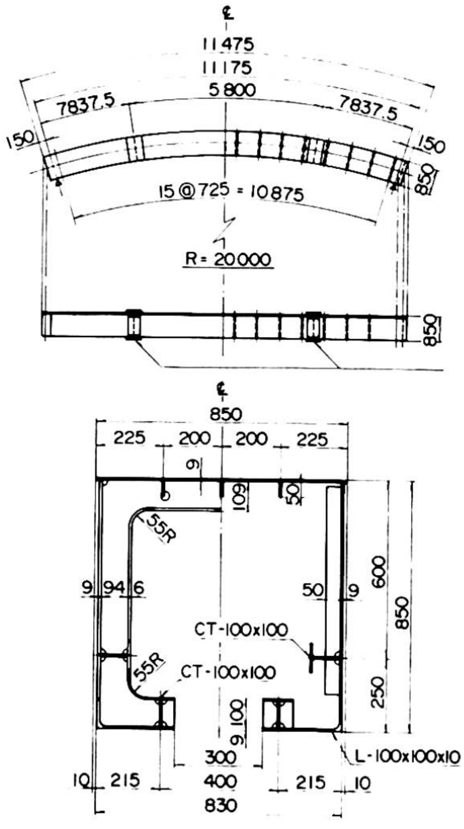


Fig. 3 Dimensions of guideway model (JIS-SS41)

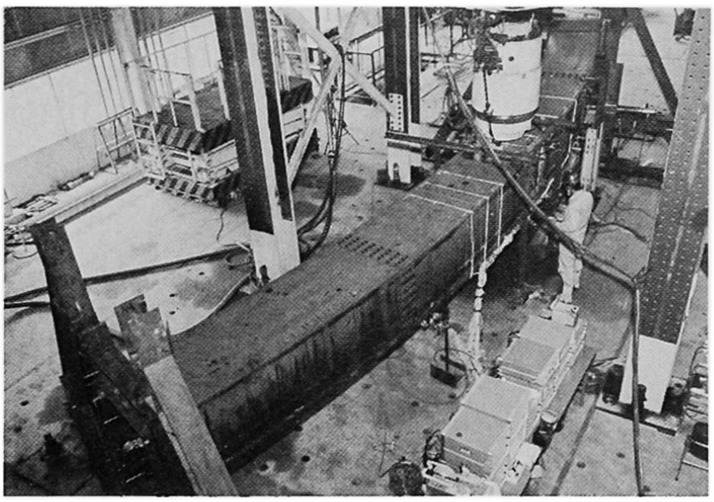


Fig. 4 General view of loading tests

The testing rig is shown in Fig. 4. The vertical jacking load was transmitted to the model through loading beam transversely placed on the parallel runway beams, whereas the horizontal jacking load through steel attachments welded to the outside of webs. The ends of the model were held within rigid restraining frames, which could be adjusted by means of steel rollers to provide both simple support condition and twisting restraint.
The instrumentation was arranged so that longitudinal and transverse stresses as well as displacements could be measured. About 400 electrical resistance strain gages were placed at various positions along the span. The strains were recorded by the automatic digital strain meters and the displacements were measured with 53 dial gages and 23 electric deflection meters.
The fatigue test was carried out by a 100-ton electro-hydraulic alternating testing machine under fluctuating bending with the same loading condition as the static vertical loading test.

3. STATIC TEST RESULTS

Since guideway cost may critically influence economic feasibility, reliable analytical techniques are needed in the design of guideway systems. Although the computer programs for the finite element method have already been developed, they are extremely expensive in practical use.
The conventional thin-walled beam theory, whether the axis of guideway is straight or curved, should be used for the basic design of the guideway beam having almost uniform cross-section, because it is able to concisely interprete overall structural behavior and is most economical. Local stresses due to the distortion of cross-section, however, have to be calculated by considering the interactions between the guideway skins and the stiffening frames. The most efficient modern method for analyzing such problem will be the finite strip method [1].
The finite strip method, however, does not permit to directly take into account the presence of transverse stiffening frames. To solve this problem the refined finite strip method [2][3] was used: the stiffening frame was represented as an assembly of finite elements. By using the flexibility matrixes of both the stiffening frames and unstiffened guideway beam, interaction forces between them were determined from compatibility equations. Finally the actual displacements and internal stresses in the whole structure were calculated.

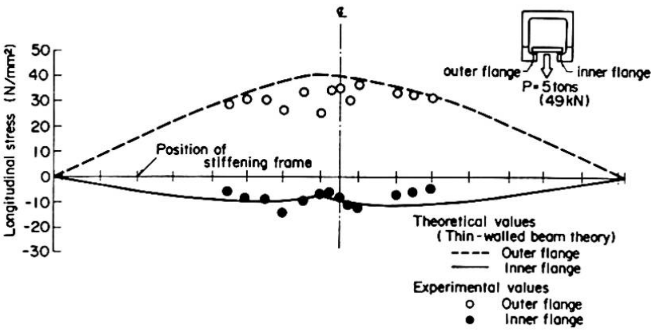


Fig. 5 Variation along span of longitudinal stresses in bottom flanges for a load applied at cross-section with stiffening frame

Both theoretical and experimental results have provided sufficient knowledges of guideway behavior, but herein a few typical results for vertical load of 5 tons (49 kN) are illustrated. In Fig. 5 the variation along the span of longitudinal stresses in the bottom flanges of parallel runway beams is plotted for a load applied at the section with the stiffening frame close to the mid-span. The corresponding distribution of longitudinal stresses at the mid-span cross-section is shown in Fig. 6. The measured stress values are in reasonably good agreement with those predicted by the thin-walled and curved beam theory.
The variation of longitudinal stresses at the same places as Fig. 5 for a load applied at the mid-span is plotted in Fig. 7. In this case the beam theory can not directly predict the local stresses induced in runway beams near the applied load, whereas the refined finite strip method can well explain

the local effect of the stiffening frames. The results indicate that it is possible, in the design of actual structures, to correct the stresses calculated from the thin-walled beam theory by adding the stresses of runway beams which are assumed to be multi-span continuous beams elastically supported by transverse stiffening frames.
In the case of horizontal loadings the general trend of stress distribution and effect of the stiffening frames were about the same for vertical ones, although the distortion of the cross-section was somewhat greater than that observed in the application of vertical loadings.

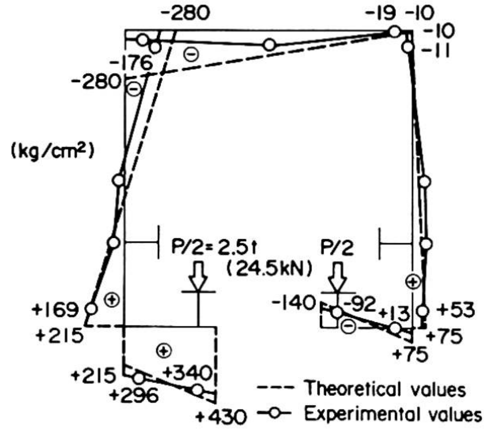


Fig. 6 Distribution of longitudinal stresses at mid-span cross-section

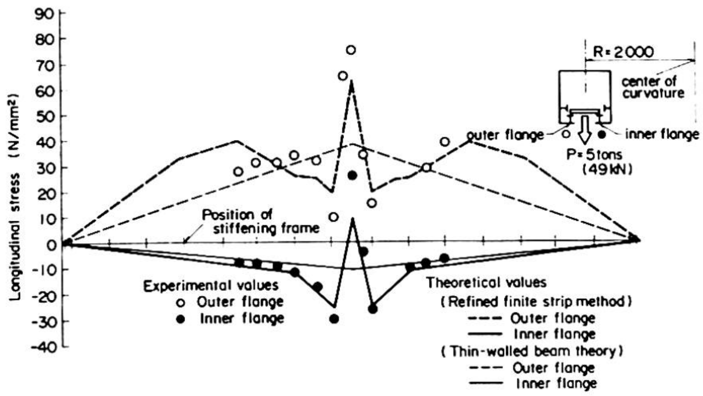


Fig. 7 Variation along span of longitudinal stresses in bottom flanges for a load applied at mid-span

4. FATIGUE TEST RESULTS

The stress range of 9.5 kg/mm^2 (93.1 N/mm^2)
was given to the fiber of outer bottom flange by the applied minimum load of 2 tons (20 kN) and the maximum load of 9 tons (88kN) under the loading speed of 1.3 Hz.
The behavior of test beam under repeated bending was quite stable without any crack initiation up to 2.3 x 10^6 loading cycles and then the fatigue test at this stress range was completed. In fatigue design calculation of guideway beams, the stresses at welded joints of runway beams with adequately located transverse stiffeners, where the fatigue strength is the minimum among structural details of runway beams, should not exceed the fatigue strength of transverse non-load-carring fillet welded joints as shown in Fig. 8 [4].
Based on the linear cumulative fatigue damage rule, the equivalent number of cycles for design stress, N_{eq}, is represented with the experimental run-out fatigue life N_{test} as

$$N_{eq} = N_{test}(P_D/P_T)^m$$

where P_T, P_D and m denote the single-axle applied load range in fatigue test, the equivalent multi-axle wheel load range and the inclination of S-N curve respectively.
The ratio P_D/P_T can be calculated as 1.57 by the aforementioned refined finite strip method and m can be read as 3.0 in Fig. 8, then the fatigue life of guideway with such structural details as the experimental model is estimated at more than 8.9 x 10^6 cycles.
For the purpose of investigation about fatigue failure mode in this model, an additional load repetition test was carried out under loading conditions of the minimum load of 2 tons and the maximum load of 11 tons. As a result, a fatigue crack caused by stress concentration and weld defects was initiated at the frame corner and propagated into stiffening frame adjacent to loading point as shown in Fig. 9. The similar cracks were initiated at other stiffening frames one after another with the decrease of stiffening effects of frames due to crack

propagations. From the results of another large-scaled guideway model tests, however, this kind of fatigue crack was proved to be avoided by perfoming full penetration without initial imperfection at flange butt welded joints or by separating the inner flanges of guide beams from stiffening frames.

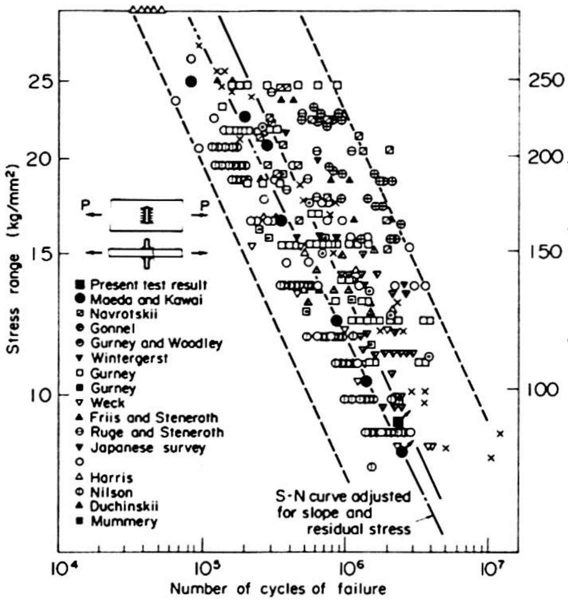


Fig. 8 S-N curve for transverse non-load-carrying fillet welded joints

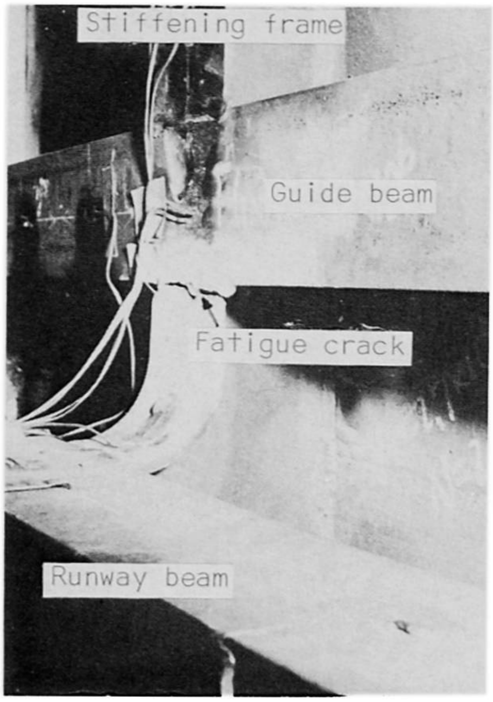


Fig. 9 Fatigue crack initiation at stiffening frame corner

5. CONCLUSION

Suspended monorail system will be one of attractive mass-transportation systems in urban areas from the view points of safety, carrying capacity, aesthetic impact, required land area, construction cost, maintenance, etc..
Guideway design for the suspended monorail must consider at least the factors of optimum size and spacing of the transverse stiffening frames, local stresses in bottom flanges beneath the wheel loads and guideway dynamics including fatigue strength.
The optimum size and spacing of stiffening frames are easily and economically determined by parametric studies using the refined finite strip method which is capable of providing an accurate representation of the behavior of guideway beams. Local stresses in bottom flanges can be predicted with reasonable accuracy by assuming runway beams to be multi-span continuous beams elastically supported by transverse stiffening frames.
From the test results, fatigue limit of guideway was estimated at more than 9.5 kg/mm^2.

REFERENCES

[1] CHEUNG Y.K.: Finite Strip Method Analysis of Elastic Slabs, Journal of Eng. Mech. Division, A.S.C.E., Vol. 94, No. EM 6 pp. 1365 - 1378, Dec. 1968.

[2] MASSONNET C.: Numerical Methods for the Linear and Non Linear Analysis of Beams, Plates and Shells, Textbook of Nagoya Lectures, pp. 27 - 31, April 1974.

[3] OTSUKA H. et al.: Analysis of Curved Girder Bridges Considering Eccentric Connection Between a Deck Plate and Girders, Proc. Japan Society of Civil Engineers, No. 259, pp. 11 - 23, March 1977.

[4] GURNEY T.R., MADDOX S.J.: A Re-analysis of Fatigue Data for Welded Joints in Steel, The Weld. Inst. Rpt., E/44/72, Jan. 1972.

IV

Barge for the Beaching of a Railway Ferry

Equipement d'abordage pour un bac ferroviaire

Anlegungsanlage für eine Eisenbahnfähre

C. CHAO
Bridge Engineer
Dept. of Shanghai Railway
Shanghai, China

K.S. SUNG
Bridge Engineer
Dept. of Shanghai Railway
Shanghai, China

SUMMARY
A set of expedient equipement for steering the ferry boat inshore is recommended in this article. It consists of three main parts: the hull, the anchoring device and the counterweights. The work done in raising the counterweight absorbs the kinetic energy produced during the beaching proceding. The design, which has gained wide approval, is characterized by the absence of heavy underwater work, high efficiency, guaranteed safety, good reliability, low cost and minimal maintenance.

RESUME
Cet article recommande un nouvel équipement d'abordage pour bac ferroviaire formé de trois parties principales: coque, système d'ancrage et contrepoids. Il s'agit d'absorber avec le travail de soulèvement du contre-poids l'énergie cinétique produite par le bac, qui passe au moment de l'abordage de l'état de mouvement à l'état de repos. Cet équipement ne nécessite pas de fondation sous-marine et se distingue d'une façon étonnante par son efficacité, sa sécurité de fonctionnement et une économie en matière d'investissements et d'entretien.

ZUSAMMENFASSUNG
In diesem Artikel wird eine neuartige Anlegungsanlage für Eisenbahnfähren empfohlen; sie besteht im wesentlichen aus 3 Teilen: Schiffsrumpf, Ankersystem und Gegengewicht. Die kinetische Energie, die beim Anlegen entsteht, wenn die Fähre aus dem Zustand der Bewegung in den der Ruhe übergeht, wird durch das Heben eines Gegengewichts aufgefangen. Bei dieser Anlage werden keine Unterwasser-Fundamente benötigt; sie zeichnet sich durch eine hohe Leistungsfähigkeit, sicheres Funktionieren und niedrige Investitions- und Unterhaltskosten aus.

A weak link occurs when a ferry is used instead of a bridge on a railway line. The ferry delays the transportation, so adequate measures must be taken to shorten the time lost. A set of equipment has been designed for this purpose as shown in Fig. 1. Three of them are reinforced concrete pile bents. No.1 and No.2 Bent are set on the front of the approach bridge to protect the bridge from the bumping of the ferry boat and to fix the forepart of the ferry boat in a fine condition for interlocking the ferry boat with the movable hinged span

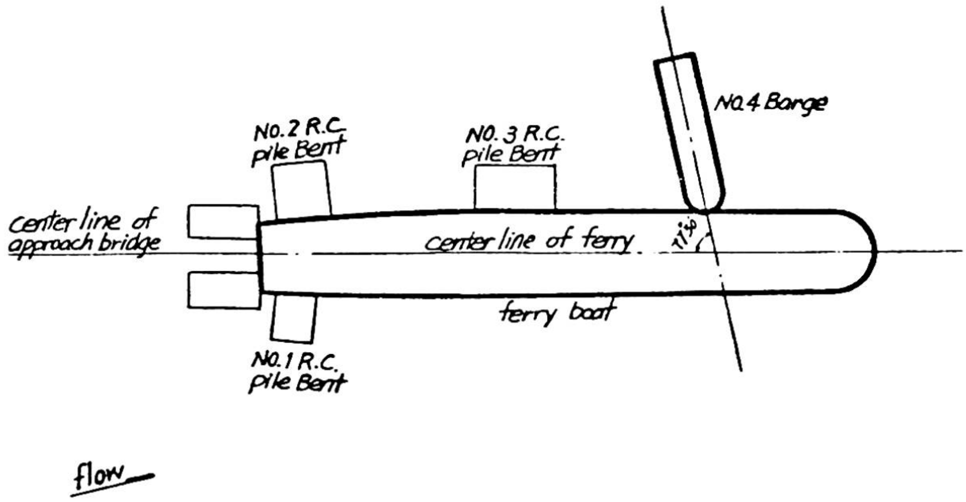


Fig. 1

of the bridge. No.3 Bent is so placed to keep the ferry boat in alignment with the center line of the bridge. No. 4 is a barge with two counterweights. The entire process of steering the ferry boat in shore is as follows (Fig.2). The ferry boat steers in shore in a slow speed at an angle about 15^{o} with the centerline of the approach bridge. The ferry boat bumps against No.4 Barge. The hitting point is about $\frac{2}{5}$ of the total length of the ferry boat measuring from the stern. The ferry boat swings shore-ward and its forepart inserts in between No.1 and No.2 Bent slowly. The function of No. 4 Barge is to absorb the energy while the ferry boat bumps against it during steering in shore and thus stops the ferry boat in a short instant. Together with the three bents, No.4 Barge stabilizes the ferry boat throughout the hauling operation of the train to or from the ferry boat. When the ferry boat leaves the shore for sailing, it is necessary to drive backward a little and to swing the forepart of the ferry boat away from the shore. No.4 Barge can also satisfy these requirements.

The barge as sketched in Fig.3 consists of three main parts:

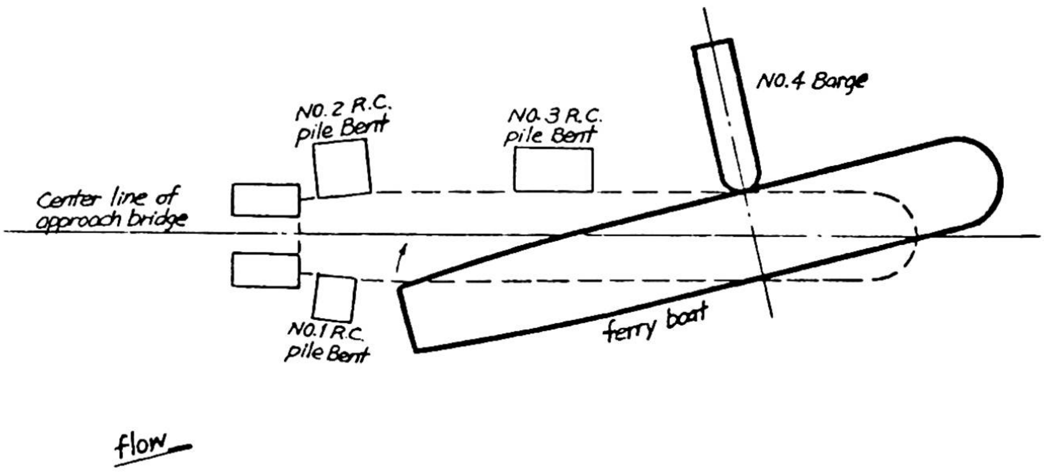
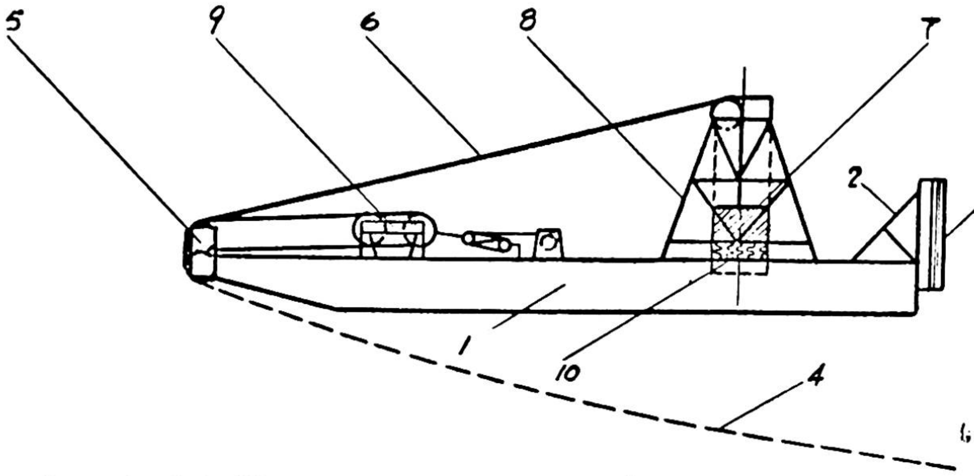


Fig. 2



1.	steel hull	2.	steel frame
3.	steel shield	4.	anchoring chain
5.	pulley block	6.	steel wirerope
7.	counterweight	8.	steel tower
9.	chain length adjuster	10.	spring bumpers

Fig. 3

1. The hull of the barge. It is a steel scow, 8 meters in width, 2 meters in depth, and 25 meters in length. On its forepart, a steel frame with a steel shield is installed. The variable positions of the board caused by loading or unloading the ferry boat determines the height of the shield. The bumping force from the ferry boat hits directly onto the shield of the barge.

2. The anchoring device. There are three pairs of anchoring chains as sketched

in Fig.4. Two reinforced concrete anchors are launched in front of the barge.

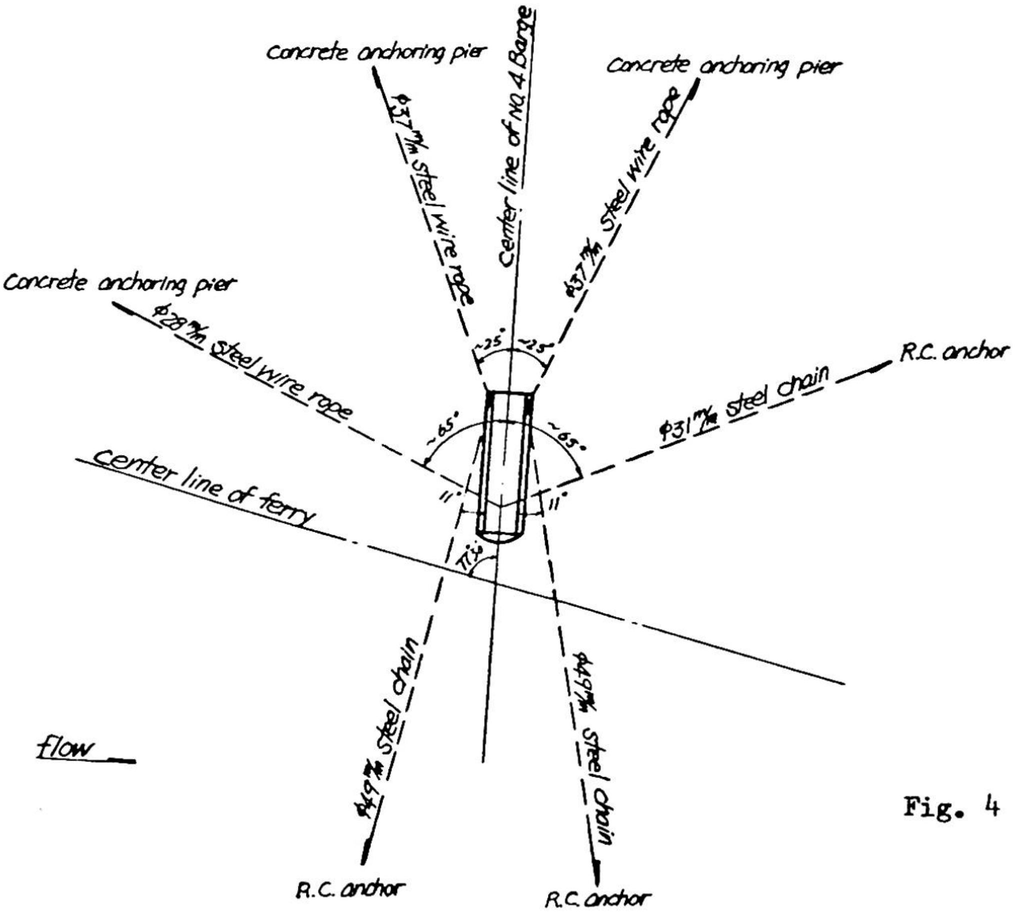


Fig. 4

Its steel chains, 49 milimeters in diameter, passing through pulley blocks at the stern of the barge are connected to two steel wire ropes. The ropes in turn passing through the pulley blocks on the top of the steel tower connect to the counterweights. When being pulled taut, the chains absorb part of the energy produced during steering the ferry boat in shore. When the energy exceeds that the chains can provide, the counterweights will then be hauled. Another pair of steel wire ropes 37 milimeters in diameter are fastened to the stern of the barge, and are anchored to the concrete anchoring piers on the shore. This pair of wire ropes keep the barge in its desired position, without them the barge would push the ferry boat away when the counterweights drop. The third pair is fastened on the front part of the barge. Toward the down-stream side a steel chain, 31 milimeters in diameter, is used with reinforced concrete anchor. Toward the upstream side is a steel wire rope 28 milimeters in diameter which is fastened to a concrete anchoring pier on the shore. These two resist the frictional force caused by the board of the ferry boat and the shield of the barge during operation.

3. The counterweights. Two counterweights each weighing 25 tons are used. They are guided by vertical channels of the steel tower. They can be lifted to a maximum height of 3 meters. When they are raised to the top of the tower, it requires an energy of 150 ton-meters. ($E=2\times25^{T}\times3^{m}=150^{T-m}$). In another words, the counterweights can absorb an energy of 150^{T-m} during steering the ferry boat in shore. The counterweights can also limit the stresses in the cables and chains, and limit the bumping force on the shield of the barge. Each cable or chain will not be loaded over 25^{T} which is the weight of one counterweight. Neither will the shield be hit by a force over 50^{T} which is the total weight of two counterweights. In this manner, the load acts upon the board of the ferry boat, the barge (including the hull, the chain, the shield, etc.) will all be limited within the designed load. Safety of these structures are thus ensured.

For each counterweight, an indicator is installed. The readings on the indicator tell the heights the counterweights being lifted. The designer limits the permissible height to 2 meters.

Besides the three main parts described above, a further device is used to adjust the length of the chains so as to keep the barge in its desired position in accordance with the variable water level in different seasons.

The design has been adopted in three railway ferries in China. Over twenty years in service, no accident causing traffic stoppage has ever occurred. This design, which has gained high approval, is characterized by the absence of heavy underwater work, high efficiency, guaranteed safety, good reliability, low cost and least maintenance.

The photo of the barge see Fig. 5.

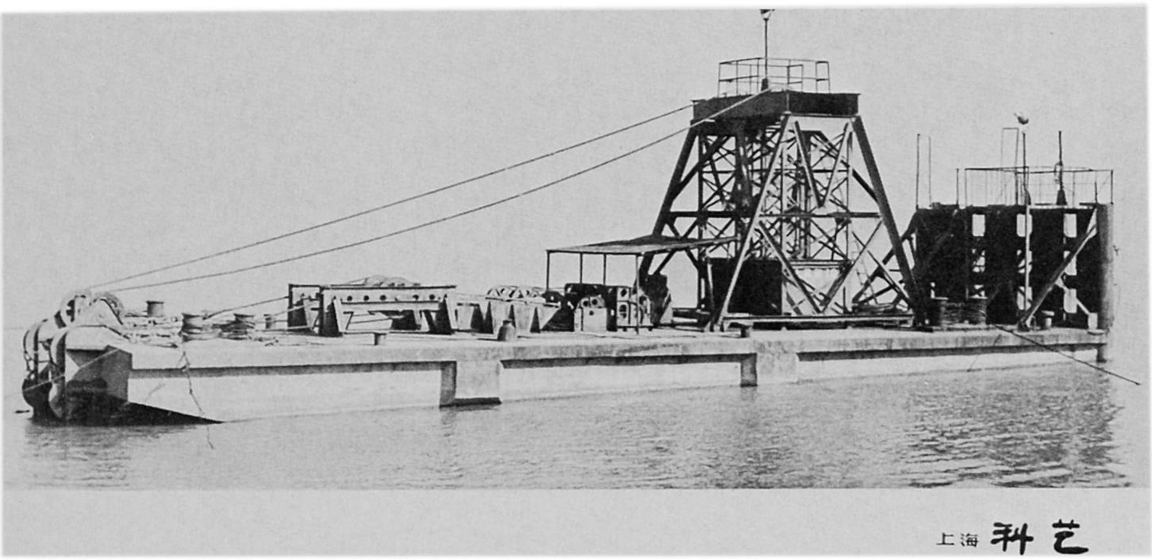


Fig. 5

Leere Seite
Blank page
Page vide

IV

Spatiostructural Bridges for Rural Roads of Mexico

Ponts spatiostructuraux pour les routes rurales au Mexique

Räumliche Fachwerkbrücken für die Landstrassen Mexikos

C. KING
Head of Technical Department D.G.S.T.
Dept. of Buildings and Public Works
Mexico City, Rep. of Mexico

J. ZAMBRANO
Head of Centro Sahop Jal.
Dept. of Buildings an Public Works
Mexico City, Rep. of Mexico

A. CALDERON
General Manager
Puentes de America S.A.
Mexico City, Rep. of Mexico

SUMMARY
Experimental engineering has proven useful in the rational design of rural bridges in Mexico. This design uses a combination of spatial trusses built up from welded light rolled steel elements, in the form of modular rectangular base pyramids, and a concrete slab, embedding the elements of the upper chord, which takes the compression stresses and acts as the bridge deck.

RESUME
La recherche expérimentale a montré son utilité dans un projet de ponts ruraux au Mexique. L'ouvrage est une combinaison d'armatures spatiales à éléments modulaires légers en acier laminé soudés, en forme de pyramides à base rectangulaire, avec une dalle en béton armé reprenant les contraintes de compression et constituant le tablier du pont.

ZUSAMMENFASSUNG
Experimentelle Forschung zeigte sich nützlich beim rationellen Entwurf der Feldwegbrücken von Mexiko. Die Brücke besteht aus räumlichen Fachwerken, die aus zusammengeschweissten, leichten Walzstahlelementen in Form von Pyramiden mit rechtwinkliger Basis aufgebaut werden und aus einer Betonplatte, welche die oberen Elemente der Fachwerke einbettet. Die Platte nimmt die Druckspannungen auf und wirkt als Brückenfahrbahn.

SPATIOSTRUCTURAL BRIDGES FOR RURAL ROADS OF MEXICO

1. FOREWORD

One of the main problems of rural roads of Mexico is the construction of bridges; considering the infrastructural restrictions, the socio-economical and environmental factors, it seems desirable to use a structure with the following characteristics:

- The construction works of the substructure and the superstructure should be made simultaneously, in order to use the maximum available skilled and unskilled man power and to reduce construction time.

- The use of scaffolding for the construction of the superstructure should be avoided to eliminate the risk of carrying-away such scaffolding by flood currents, and to allow a continuous work-period throughout the whole year.

- The design of a self-supported superstructure allowing to place an adequate concrete slab deck, with an economical and simple formwork.

- The design of a superstructure of minimum weight, but with an adequate rigidity to support its own weight and the live operating loads, as well as to enable an easy and safe launching operation.

- The design of a structure that allows an easy access for adequate supervision during the construction stage and for maintenance inspections during operation.

- The possibility of enlarging the bridge with future traffic demands.

- The possibility of making bridge repairs without traffic interruptions.

2. STRUCTURAL DESIGN

Among the various alternatives studied by the Mexican technicians, it was found that superstructures of spatial welded steel elements could conform the requirements, as they allow to profit from the steel and concrete characteristics in the best way.

3. DESCRIPTION

The superstructure design consists of a tri-dimensional truss constituted by multiple steel linear elements, geometrically arranged in the position of rectangular base pyramidal edges, opposed to each other and joined by their vertex. With this configuration, a very light self-supported structural system is obtained with a considerable tri-dimensional rigidity, which enables to cover relatively large spans, avoiding the need of scaffolding for its erection.

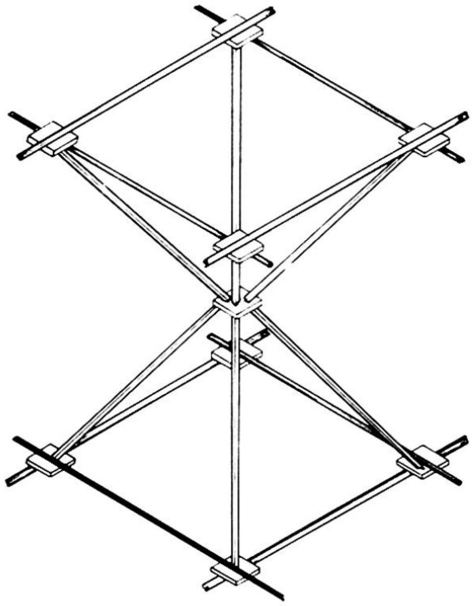


Fig. 1 Basic Modular Element of a Rural Spatiostructural Bridge (Sketch)

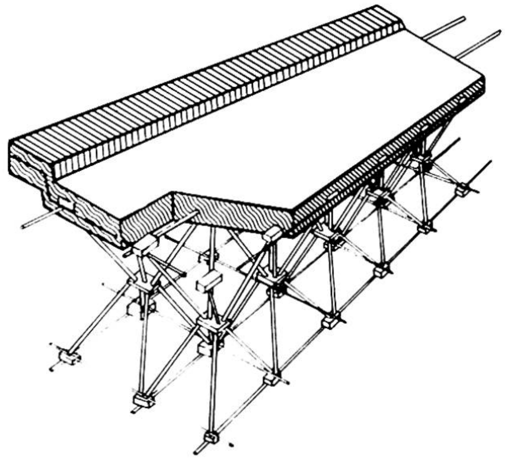


Fig. 2 Isometric Detail of a Rural Spatiostructural Bridge (Sketch)

In the upper part of the truss, a concrete slab of approximately 15 cm deep is casted, embedding the elements that form the upper chord, which collaborates in taking the structural compression stresses, and besides acts as the floor of the bridge, as shown in the above Figures 1 and 2.

4. MANUFACTURING AND ERECTION

To manufacture these trusses, laminated structural angles are used for the diagonals and corrugated steel bars for the upper and lower chords. The manufacturing is made in framed sections of adequate dimensions and weight to facilitate their transportation from the workshop to the jobsite. See Table I.

Nº	ELEMENT IN cm	SECTION	STEEL TYPE
1	3.81 x 3.81 x 0.32		A - 36
4	3.18 x 3.18 x 0.32		
6	2.54 x 2.54 x 0.32		
I	2.54 x 2.54 x 0.32		
II	3.18 x 3.18 x 0.32		
2	3.18 x 3.18 x 0.64		
3	3.18 x 3.18 x 0.64		
5	3.18 x 3.18 x 0.32		
7	2 ϕ # 8		TOR-60
8	2 ϕ # 10		
9	1 ϕ # 10 + 1 ϕ # 12		

Table I Typical Sections for a Rural Spatiostructural Bridge

The sections are taken to the riverside and are there assembled for a further launching into their final position on the substructure. For the correct assembly, adequate joints have to be provided at the end elements of the framed sections: the continuity of the sections is obtained by extending and lapping the steel of the upper and lower truss chords, which are welded when assembling the elements. The casting of the upper deck is made by means of a light weighted formwork suspended from the upper mat of the structure. See Figures 3, 4, 5 and 6.

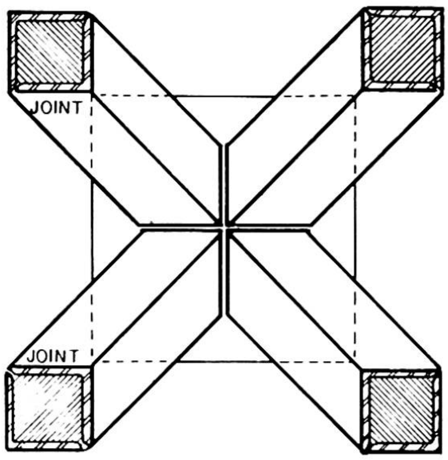


Fig. 3 Upward View of an Intermediate Node (Sketch)

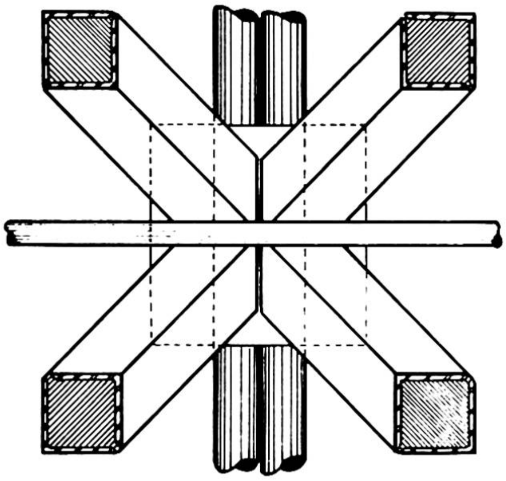


Fig. 4 Downward View of a Lower Node (Sketch)

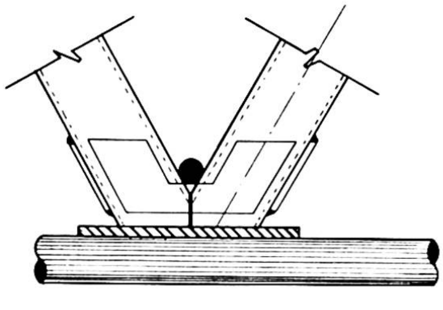


Fig. 5 Longitudinal View of a Lower Node (Sketch)

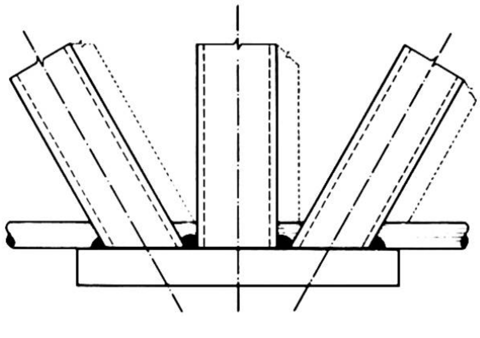


Fig. 6 Detail of a Lower Bearing Node (Sketch)

5. DESIGN ANALYSIS

For the design verification, the superstructure is analyzed as a spatial truss, in which the elements are subject to simple tension and compression stresses. It is considered that the concrete embedding the upper mat helps to prevent buckling of the horizontal members subjected to compression and is used as a floor for the passage of the moving load.

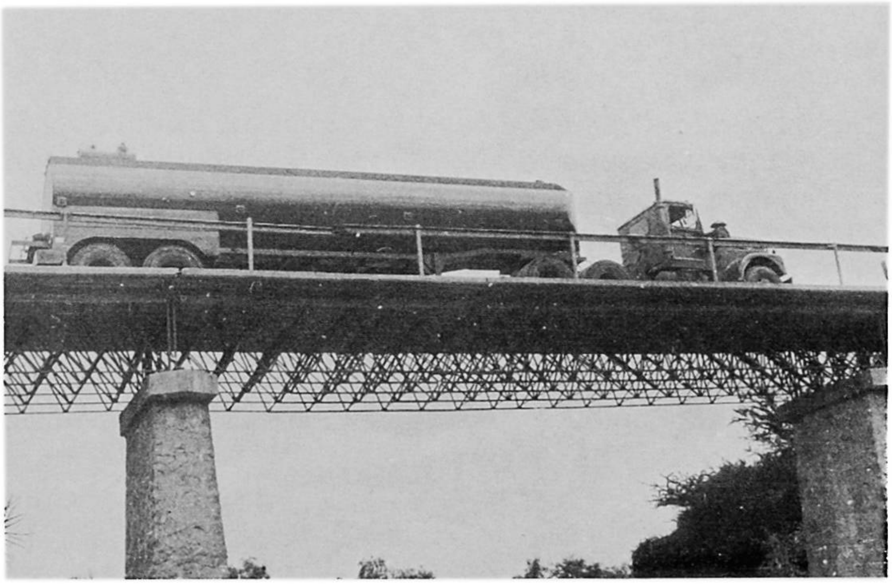
In the analysis, it is implicitly supposed that the load-displacement ratio is linear and also that the displacements produced are relatively small. When a computer is available and a more accurate verification is desirable, more complex procedures may be applied, such as displacement or force methods, on which the matrix analysis of structures is based.

6. EXPERIMENTAL VERIFICATION

As no data concerning the results of any previous experience related with the behaviour of bridges with this type of structures were available, some of them have been instrumented with electrical strain gages of SR4 type. In some cases, the instrumentation has started at the cutting and welding stage of the steel elements to register the historical evolution of stresses in these elements, as well as to know the influence of the variations produced in the stresses due to the effect of labour in the assembling and erection procedures.

Once completed the construction, and before the instrumented bridges are put into operation, some load tests have been carried out with heavy loaded trucks parked in the most critical positions, as shown on Figure 7.

Fig. 7 Load Test



7. RESULTS

In general, the results show that measured stresses are lower than those calculated and that they are far from reaching the allowable design stresses, which implies that the adopted hypothesis for the calculation are adequate and secure. On the other hand, the low registered values for the measured stresses suggest the convenience of optimizing the design; for this reason, an experi

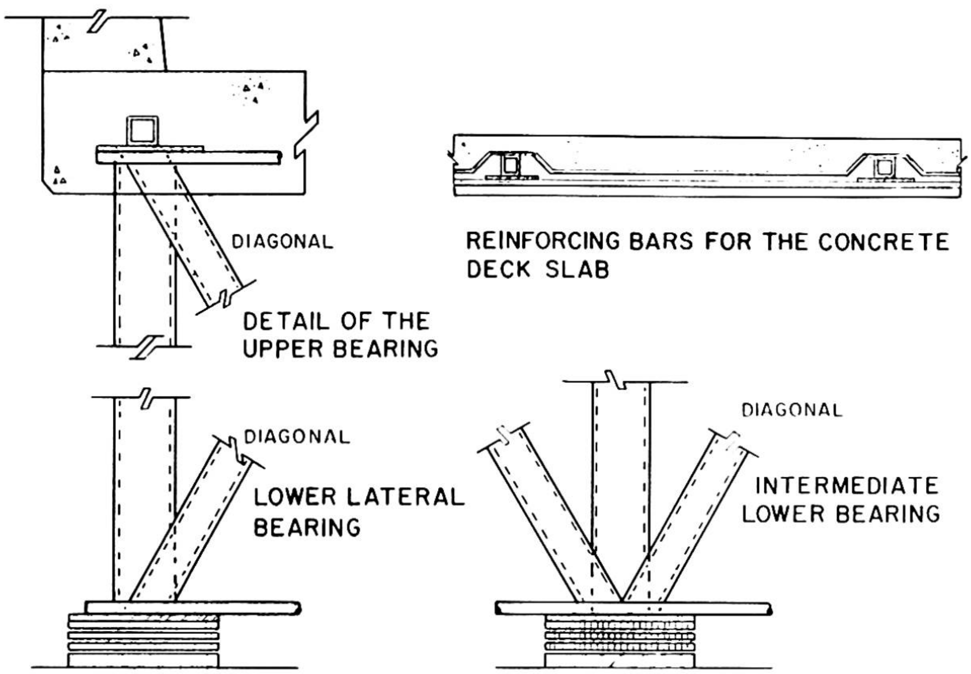


Fig. 8 Bearing and Reinforcing Details

mental engineering research is already being carried out, by means of physical models, in order to improve the calculation hypothesis. It has to be kept in mind, nevertheless, that any optimization in the design requires of more accurate supervision and quality control during the construction stage.

8. CONCLUSIONS

The successful results of the Mexican experience, make clear that the uncertainties created by rational designs in developing countries may be evaluated by means of an adequate application of experimental engineering, because with an instrumented observation of models and prototypes, it is possible to anticipate in a true and reliable way the actual behaviour of the designed constructions.

For these reasons, we believe that experimental engineering is one of the most promising resources for the technological independence of the developing countries.

ACKNOWLEDGEMENTS

The authors are greatly pleased to express their gratitude to the authorities of the Secretariat of Human Settlements and Public Works, and particularly to the Secretary, Arq. Pedro Ramírez Vázquez, the Undersecretary of Public Works, Ing. Rodolfo Félix Valdés, and the General Director of Technical Services, Ing. Gustavo del Río San Vicente, for their valuable assistance and support in the preparation of this paper.

IV

Steel Reticulated Plates in Industrial Buildings

Plaques à treillis en acier pour la construction industrielle

Räumliche Fachwerke aus Stahlplatten im Industriebau

MAURO MEZZINA
Ass. Prof.
Universita di Bari
Bari, Italy

GIUSEPPE PRETE
Prof.
Universita di Bari
Bari, Italy

ANTONIO TOSTO
Ass. Prof.
Universita di Bari
Bari, Italy

SUMMARY
This paper concerns the application of special structures such as steel reticulated plates in industrial buildings. Their typological aspects and technical characteristics are examined. For the solution of technological and erectional problems, an original constructional system is proposed, together with some significant experiments.

RESUME
On examine les applications des plaques réticulaires métalliques dans les bâtiments industriels et leurs aspects typologiques et techniques. On propose ainsi un système de construction original pour la solution de problèmes technologiques et de montage, avec la description de quelques essais significatifs qui ont été réalisés.

ZUSAMMENFASSUNG
Der Artikel behandelt die Anwendung spezieller Strukturen, z.B. räumliche Fachwerke aus Stahlplatten im Industriebau. Ihre typologischen Aspekte und die technischen Eigenschaften werden untersucht. Es wird ein neues Konstruktionssystem zur Lösung der technologischen Probleme sowie der Probleme bei der Montage dargestellt — zusammen mit einigen der durchgeführten Untersuchungen.

1. TYPOLOGY AND USE

The most important applications of metallic space structures in civil and industrial buildings are reserved to flat or slightly curved reticulated plates. They are double (or triple) layered grids made up of two (or three) parallel and

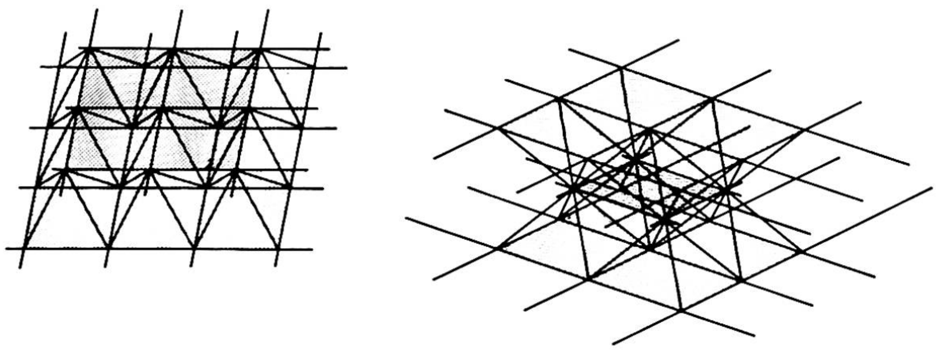


fig. 1

modular networks interconnected by vertical and diagonal web members and loaded normally to their plane (fig. 1).

The base networks, multi-directionally framed in their plane, can be perfectly overlapped or joggled, geometrically identical or different. The most frequent network schemes are shown in fig. 2. In particular we define the bi-directional grid as <u>rectangular</u> if the members cross at 90° and are side parallel; otherwise <u>diagonal</u> if they are slanted on their support line. Sometimes networks, lightened by taking away suitable members and nodes, are used. The most significant recent development of this typology is represented by the bi-directional <u>crossed</u> grid (fig. 3): it is obtained by diagonalizing one of the layers (top or bottom) and leaving the other as a rectangular one (<u>diagonal on square</u> or <u>square on diagonal</u> grid).

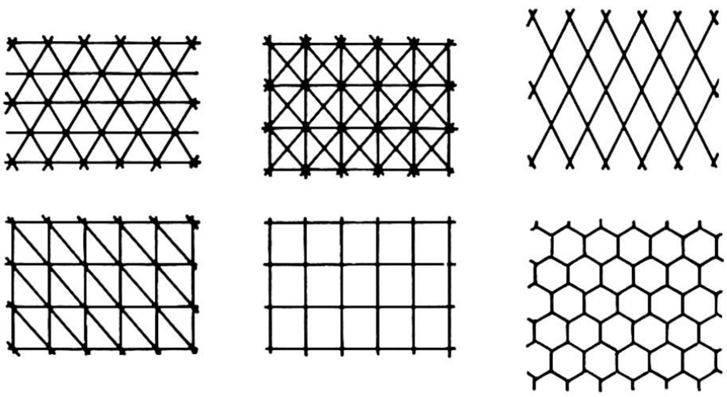


fig. 2

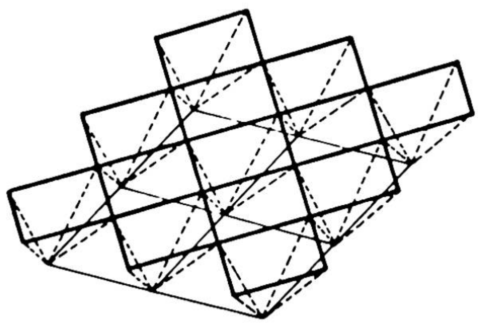


fig. 3

The reticulated plates are characterized by the favourable effect of the force distribution; these are in fact almost exclusively axial since the loads are normally applied on the nodes. Each element of these structures performs, in optimal conditions, a lot of static and constructional functions: the members are, at the same time, principal beams, secondary beams and bracings.

Presently their major use is reserved for the covering of special large span constructions: theatres, exibition centres, sport pavillions, churches, hangars, etc; but also for flooring of normal multistoried civil and industrial buildings with small spans. In this case their use is often joined with prefabricated collaborating r.c. panels.
Their depth varies between 1/20 and 1/30 of the span, changing with the loading, the geometric scheme and the node distance. The open space between the two layers enables horizontal technical services contained within the volume of the structure (1).

Regarding their analysis, we note that the computer program use has presently eliminated any difficulty, while the handle calculation is now usually reserved for the pre-dimensioning, in which they can be analyzed as continua, applying the plate or shell analogies (2).

2. CONSTRUCTIONAL PROBLEM: THE PREMIT SYSTEM

The great interest in reticulated plates is reflected in the large number of commercial construction systems recently developed throughout the world. Among the well-established and already well-known systems we mention the Mero, Oktaplatte, Unistrut, Unibat, Nodus, etc. These systems are different both for the construction method and for the node assembling. The "node" is the key element of these structures. It must be sufficiently resistent for trasmitting the member forces, undeformable under static and dynamic loadings and easy to construct and erect. Several of the systems presently used, even if optimal from a static and aesthetic point of view, nevertheless present some economic disadvantages. These are due to the complexity of the assembling nodes, requiring expensive special pieces and made with sophisticated technological processes, or to the necessity of field weldings.

We shall explain here our solution of the problem already applied in some important buildings, after suitable experimental tests (3).
The system, named PREMIT, uses the same elements and procedures of the traditional carpentry buildings with which we believe it competitive from an economic point of view. It is based on the shop construction of two standard structural components (fig. 4): the <u>diagonal member</u>, typically made up by a tubular section, and the <u>chord member</u> (top or bottom), preferably made up by tubular section, but also by different sections. Both elements have a suitable equipment welded at their ends ("niches" or "small-plates"). The structural assembling follows then immediately (fig. 5): the grid generic node is obtained by the juxtaposition of four diagonal and chord members. The connection is ensured, without any eccentricity, by means of only four H.S. friction bolts.
The proposed system, according to modular coordination criteria of structural components with selected unitary dimensions, serves for the construction of reticulated plates with <u>rectangular bi-directional</u> and <u>diagonal</u> schemes (with top

and bottom joggled networks) or with derived lightened schemes; with some simple modifications this system can be applied also to the crossed grid of fig. 3.

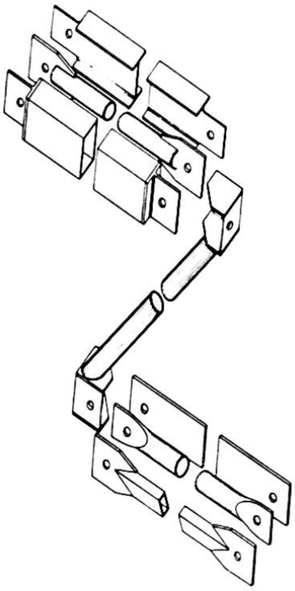


fig. 4

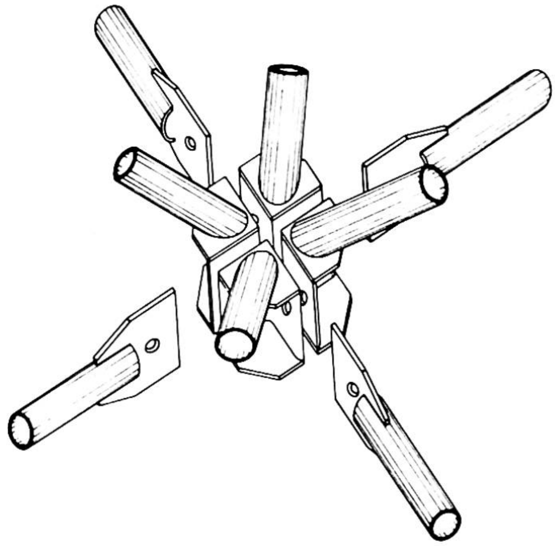


fig. 5

Its principal advantages are as follows:

- The node is obtained without the aid of any special loose part (as plate, ball, diaphragm, etc.) to hold the junction; its mechanical function is exclusively obtained through its elements: "bolts", "niches" and (optional) "small-plates".
- All the structural components of this system can be completely prefabricated in a workshop, including all its welding operations.
- For the member realization, eather the use of tubular or open section profiles is possible.
- The final assembling of the structure is carried out in the working-place with only four friction bolts per node, in which eight members converge.
- It is possible to partially neutralize the secondary moments in the chord members, due to loads directly applied on them, marking-out the structural elements with a pre-fixed and suitable eccentricity between the geometric and the bolt axes.
- Due to its special simplicity, the node can be easily calculated because of a mathematical schematization, which permits the static dimensioning of all its components (4).

3. LABORATORY AND DESIGN EXPERIENCES

Suitable laboratory experiences have been performed both on node models (fig. 6) and on double layered grid prototypes (fig. 7), purposely realized, for verifying the static and functional validity of the PREMIT system. The experimental

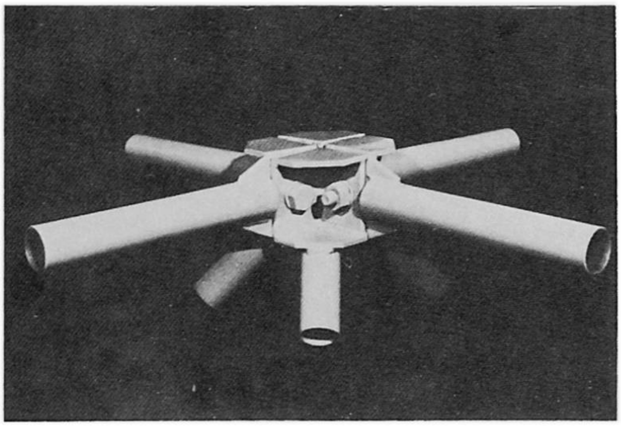


fig. 6

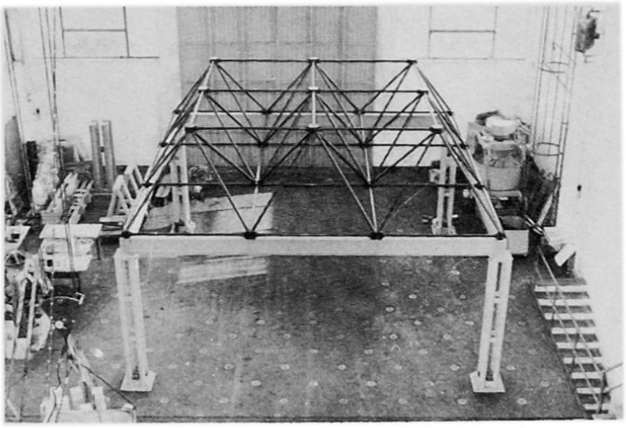


fig. 7

		COMPARISON VALUES :	THEORET.	EXPERIM.	VARIATION %
MODEL NODE TESTS	UNIAXIAL TENSION	SLIP LOAD	71196 N	88260 N	+24
		SLIP SAFETY FACTOR	1.25	1.55	+24
		COEFFICIENT OF FRICTION	0.30	0.372	+24
	BIAXIAL TENSION	SLIP LOAD	54770 N	68647 N	+25
		SLIP SAFETY FACTOR	1.25	1.56	+25
		COEFFICIENT OF FRICTION	0.30	0.407	+36
PROTOTYPE GRID TESTS	INSTABILITY LIMIT STATE	CRITICAL MULTIPLIER	3.23	4.00	+24
		BUCKLING SAFETY FACTOR	1.50	1.85	+23
		MAX. DEFLECTION	5.46 mm	6.65 mm	-22
	MECHANISM LIMIT STATE	ULTIMATE MULTIPLIER	7.33	7.50	+2.3
		COLLAPSE SAFETY FACTOR	1.51	1.55	+2.6
		MAX. DEFLECTION	9.66 mm	9.80 mm	-1.4

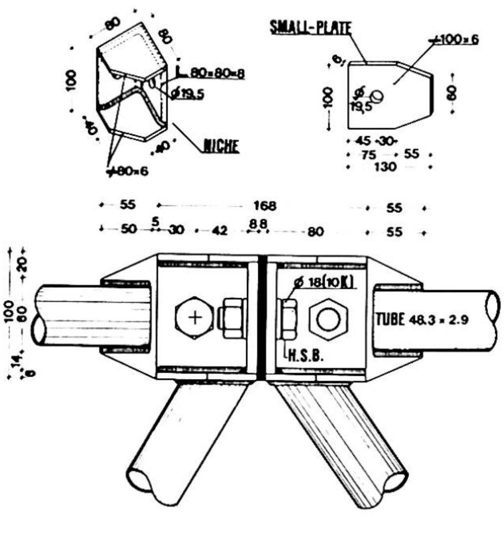


fig. 8

tests have always been carried out up to the collapse and the results, found very interesting, have been reported in (3). Referring to the node in fig. 8, we have here enclosed a table in which it is possible to note the slight variations between the theorical and experimental values.

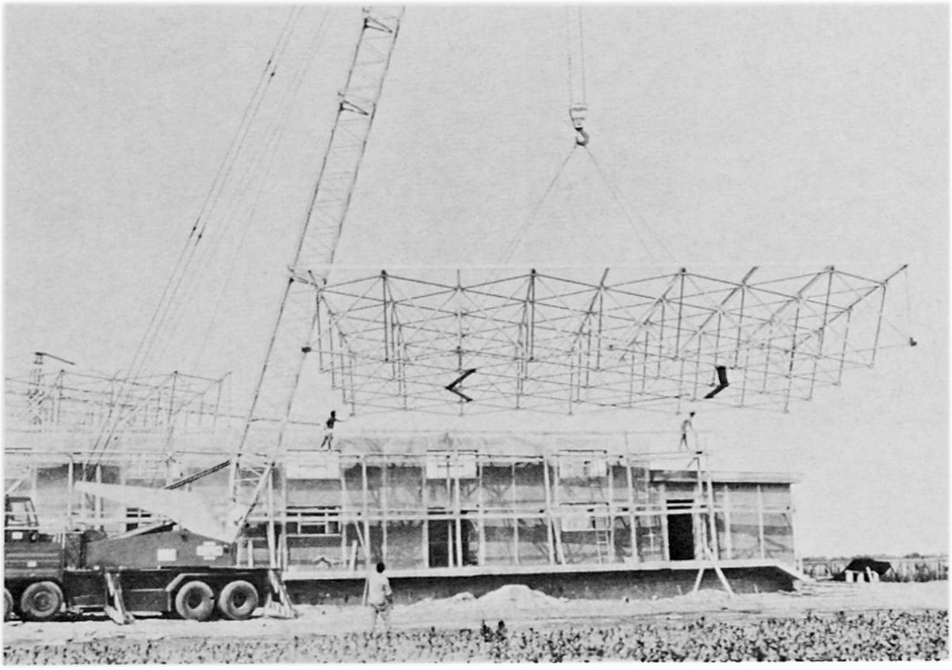


fig. 9

After these conforting experiences we have applied our PREMIT system in some industrial building coverings (5), using the "lift-slab" method for the erection, with excellent economic results (fig. 9).

REFERENCES

1. A. PADUART: Structures spatiales discontinues. C.B.L.I.A., Bruxelles, 1973.
2. G. PRETE: Analisi al continuo di grigliati spaziali multipli. Atti dell'Istituto di Scienza delle Costruzioni dell'Università di Bari, n. 115, 1978.
3. D. MITARITONNA and G. PRETE: Proposta e sperimentazione di un nuovo sistema di connessione nodale per grigliati spaziali in acciaio. Costruzioni Metalliche, n. 4, 1977.
4. G. PRETE and D. MITARITONNA: Sistema costruttivo PREMIT: criteri di calcolo e modelli applicativi. Atti dell'Istituto di Scienza delle Costruzioni della Università di Bari, n. 116, 1978.
5. M. MEZZINA, G. PRETE and A. TOSTO: Coperture spaziali per stabilimenti enologici in Puglia. Costruzioni Metalliche, n. 5, 1978.

IV

Netzkuppel mit 236,5 m Spannweite

Network Cupola of 236,5 m Diameter

Coupole à treillis d'un diamètre de 236,5 m

N.P. MELNIKOV
Prof. Dr.
ZNIIProjektstalkonstrukzija des Gosstroj
Moskau, UdSSR

W.A. SAWELJEW
Kandidat d. Techn. Wissenschaften
ZNIIProjektstalkonstrukzija des Gosstroj
Moskau, UdSSR

ZUSAMMENFASSUNG
Im Vortrag wird die stählerne Konstruktion einer Netzwerkkuppel mit 236,5 m Durchmesser und 114 m Höhe vorgestellt, die im „ZNIIProjektstalkonstrukzija" entworfen wurde.

SUMMARY
Briefly the structure of a steel network cupola of 236,5 m diameter and 114 m high designed by the Institute „ZSNIIproektstalkonstructsiya" is described.

RESUME
Le rapport contient une brève description d'une coupole à treillis en acier d'un diamètre de 236,5 mètres et d'une hauteur de 114 mètres conçue par „TSNIIproektstalkonstructsiya".

Ende 1978 wurde es im Vorort von Moskau Istra mit der Montage der Metallkonstruktionen für das Hauptlaboratoriumsraum des W.I. Lenin-Allunionsinstituts für Elektrotechnik begonnen, das für Ausrüstungsprüfungen der neuen, kräftigen, elektrischen Fernleitungen bestimmt ist.

Das Gebäude ist als gitterförmige Rotationsschale ausgeführt, deren Form einem abgeplatteten Ellipsoid mit Äquatorialdurchmesser von 236,5 m ähnlich ist. Der Äquator ragt 23,2 m über der Höhe der Fussbodenoberkante. Der Durchmesser des Gebäudes in der Gründung beträgt 231,7 m. Die Höhe des Gebäudes im Mittelpunkt beträgt 118,4m*(Abb. 1).

Die Schale hat eine inkonstante Flächenkrümmung und ist als Stabnetz mit Zellen ausgeführt, die die Form eines gleichschenklichen Dreieckes haben, wobei die Gründungen dieser Dreiecke auf horizontalen Ringen ruhen (Abb. 2).

Die eine Raumstruktur bildenden Gerippestäbe sind zweistrangig; sie werden als Parallelträger ausgeführt. Die Gurte bestehen aus gepaarten Winkelprofilen. Die Querschnittshöhe beträgt 2,5 m. Der äussere Gurt besteht aus zwei nichtgleichflanschigen Winkeln 160 x 160 x 9, durch breite Flanschen gebildet, und der innere Gurt aus gleichflanschigen Winkeln 160 x 160 x 10. Das Material der Gurte ist der 09 Г2С-12-Stahl. Das Gitter wird aus elektrogeschweissten Röhren 60 x 2,5 und 83 x 2,8 ausgeführt.

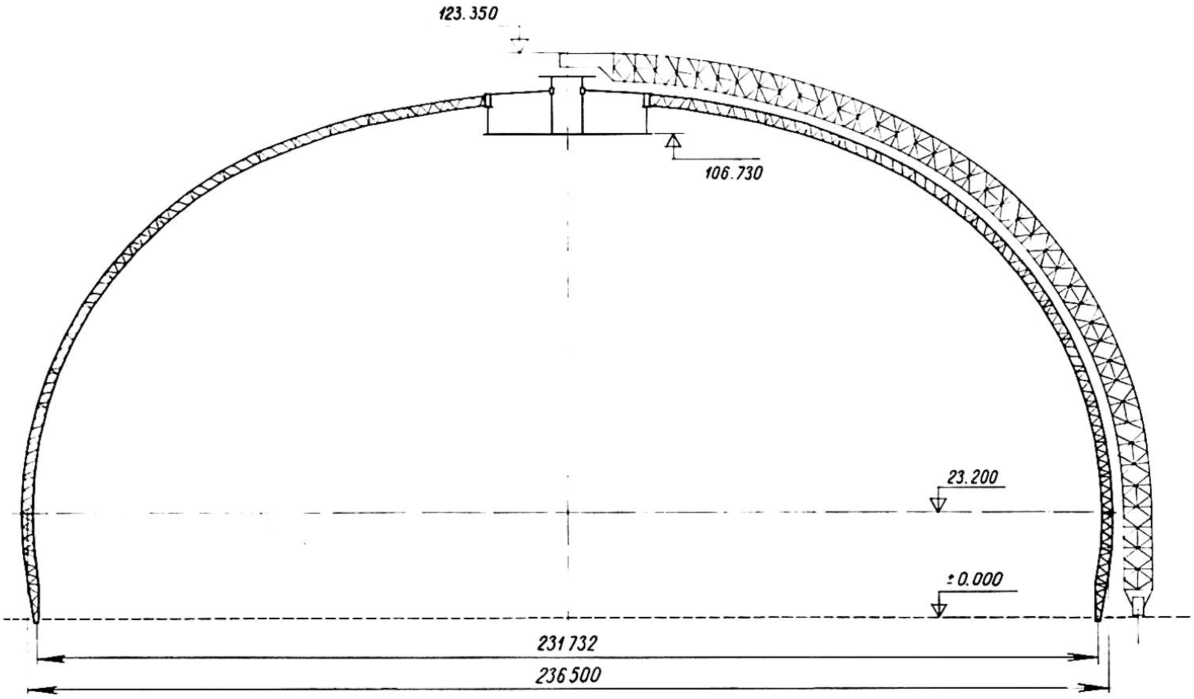


Abb. 1 Querschnitt des Gebäudes

* Angeführte Masse beziehen sich auf die mittige Schalenfläche.

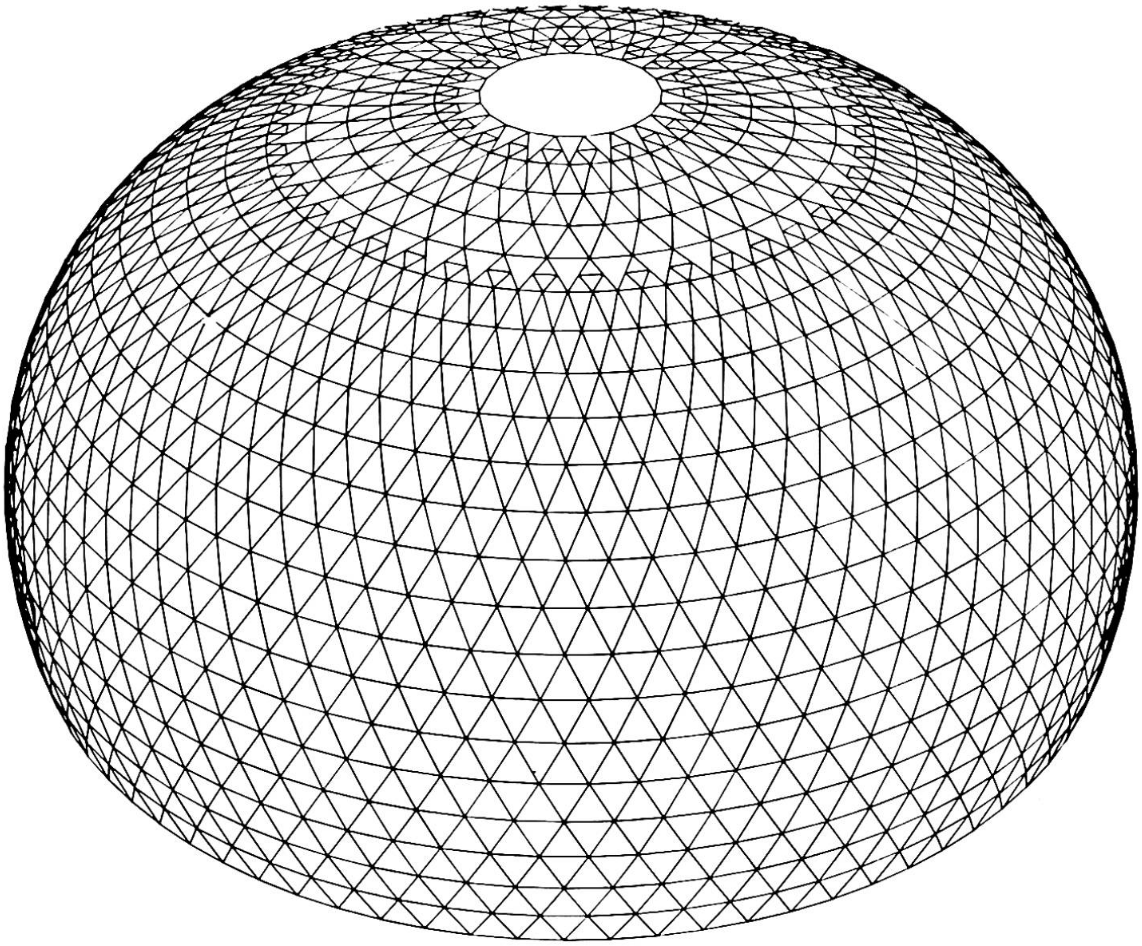


Abb. 2 Ansicht der Gerippeschale

Laut dem Projekt wird das Schalgerippe auf 83 Einzelstützen gelenkig aufgelagert.

An den äusseren Gerippegurten wird eine Membrane aus 1,5 mm dickem Rollenstahl 10ХНДП angeschweisst. Die Membrane nimmt Wind- und Schneebelastungen auf und dient gleichzeitig als Hüllenkonstruktion.

Die Begrenzung des ersten, unteren Geschosses wurde aus profiliertem Stahlbelag projektiert, der an Ringpfetten mit selbstschneidenden Schrauben befestigt wird. Die Wasserundurchlässigkeit wird durch die Überlappung der Verkleidungsplatten auf eine Breite von 100–200 mm bei der Neigung > 25% erreicht. Für den oberen Gebäudeteil (die Neigung < 25%) wird eine einlagige Polymerschicht mit einer Dicke von 1,5 mm vorgesehen.

Von innen werden an Gerippestäben Hängedeckenplatten befestigt. Die Hängedecke erfüllt 4 Funktionen: Abschirmen des Innenraums, Schal-, Wärme- und Feuerschutz. Die Abbildung 3 zeigt den Querschnitt einer Hängedeckenplatte.

Im Kuppelscheitel befindet sich das technologische Raum von 34 m Durchmesser und 900 m² Bodenfläche.

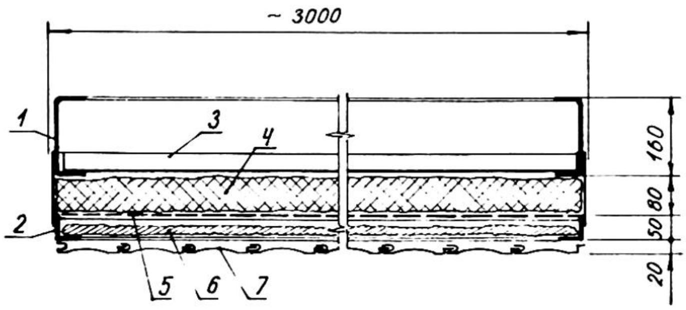


Abb. 3 Querschnitt der Hängedeckenplatte
1 - Biegeprofil aus U-Stahl Nr. 16
2 - gleichschenkliger Winkel
3 - profilierter Belag h = 40
4 - Mineralwatte
5 - Stahlnetz
6 - Glasfaser
7 - Bekleidung aus profiliertem Al-Profil

Das Gebäude ist mit 1x100t-, 4x25t und 62x5t-Hebeeinrichtungen eingerichtet, die auf innerer Fläche gleichmässig verteilt sind.

Die Wartung der Gebäudeoberfläche wird durch eine Sondereinrichtung als Halbbogen vorgesehen, die als räumliche Rohrkonstruktion ausgeführt ist und deren Unterteil auf Karren mit Fahrantrieb auflagert, die ihrerseits auf einem Bahngleis um das Gebäude herum fahren. Der Scheitel des Halbbogens wird zum Gebäudeoberteil gelenkig befestigt. Im Innern des Halbbogens stehen ein Lastaufzug und Treppen zur Verfügung.

Der Halbbogen lässt die Elektroenergie, das Wasser und die Druckluft zu einem beliebigen Punkt der Oberfläche anleiten, was eine Möglichkeit gibt, den Dachbelag zu besichtigen, zu reinigen, anzustreichen und auszubessern. Der Halbbogen sichert die Materialzustellung bis zu 1 t Gewicht und das Leutetransportieren zu einem beliebigen Punkt der Gebäudeoferfläche. Diese Einrichtung erfüllt gleichzeitig die Funktion des Schneeräumers und der Feuerlöschung.

Die Herstellung aller Hauptelemente der Raumstabschale (zweistrangige Stäbe, Knotenbauteile, Verbände usw.) erfolgt fabrikmässig.

Die Konstruktionsteile werden auf der Baustelle durch Grossblockmontage in zwei Phasen zusammengestellt. In erster Phase werden die 3 ebenen Parallelträger aus einzelnen Winkeln, die gleichzeitig eine Hälfte der Gerippestäbe bilden, zu räumlichen Dreieckzellen vereinigt.

In zweiter Phase werden Dreieckzellen zu Montageeinheiten mit 2-4 Zellen für eine Einheit vereinigt. Die Einheiten stellen räumliche Parallelogramme oder Trapeze vor und sind Teile der stufenförmigen Unterteilung der Schalenoberfläche.

An die Aussenfläche der Montageeinheit wird eine Stahlmembrane angeschweisst, an die Innenfläche werden Hängedeckenplatten befestigt. Das gesamte Gewicht der Montageeinheit beträgt ~ 10 t.

Die Konstruktionsteile werden zu Grossblöcke durch das Montageschweissen zusammengestellt.

Die Montageeinheiten und ihre Verbindungsknoten wurden unter der Bedingung entworfen, dass die Montage im Freivorbau verwendet werden konnte, d.h. das Anstücken der Schale erfolgt stufenweise ohne Hilfsstützen in Richtung von der Gründung zum Oberteil. Alle Montagestösse sind durch HV-Bolzen verschraubt. Die Abbildung 4 zeigt einen Verbindungsknoten.

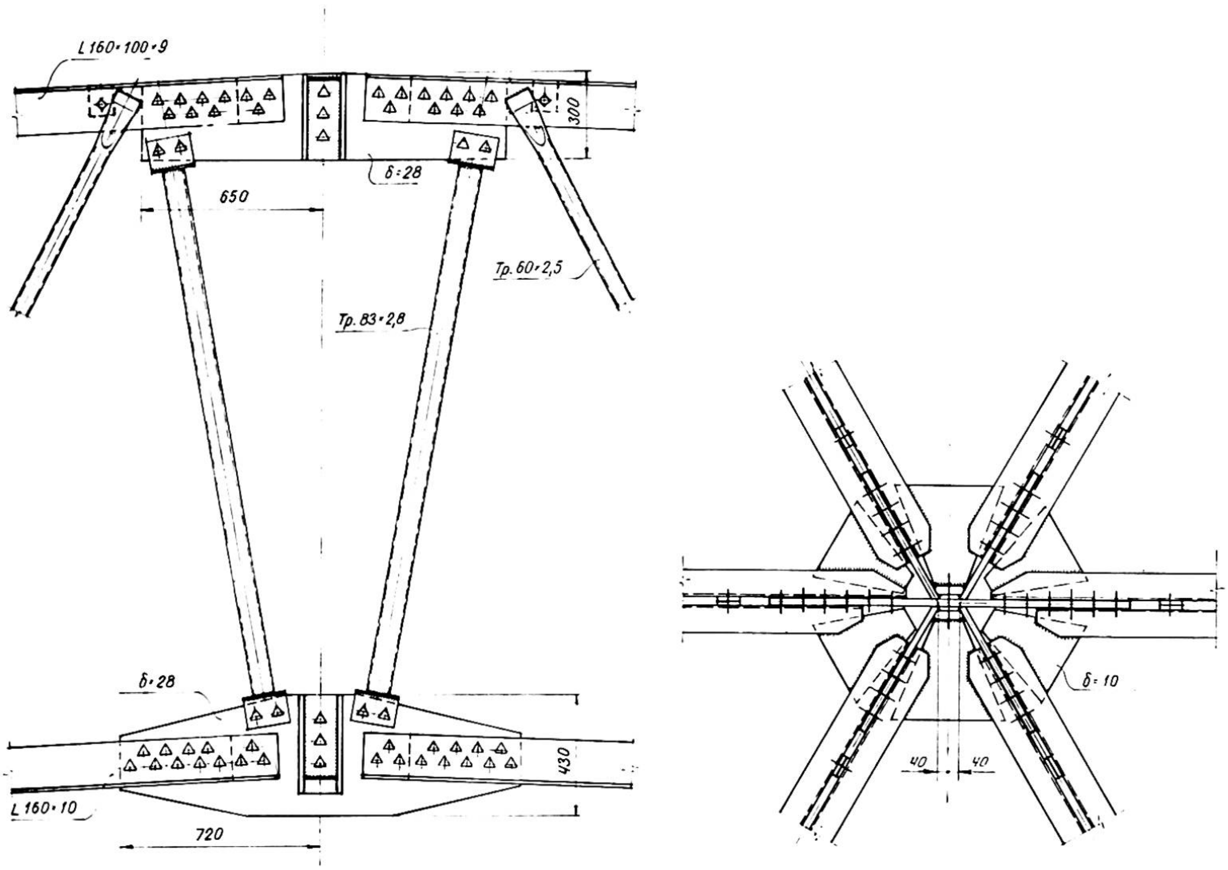


Abb. 4 Verbindungsknoten der Gerippestäbe

Unter allen Gerippeknoten werden Montagebühnen angeordnet, die durch Ringübergänge und Treppen verbunden sind, die ihrerseits für die Bedienung der Förderanlagen weiter genutzt werden.

Die Konstruktion des zentralen technologischen Raums soll in der Nullhöhe vollmontiert und in die Entwurfshöhe mittels Winden aufgehoben werden, die am schon montierten Gerippeteil befestigt sind.

Der Stahlverbrauch für Haupttrag- und Hüllkonstruktionen des Gebäudes (inklusive des Membranbelags, des technologischen Raums

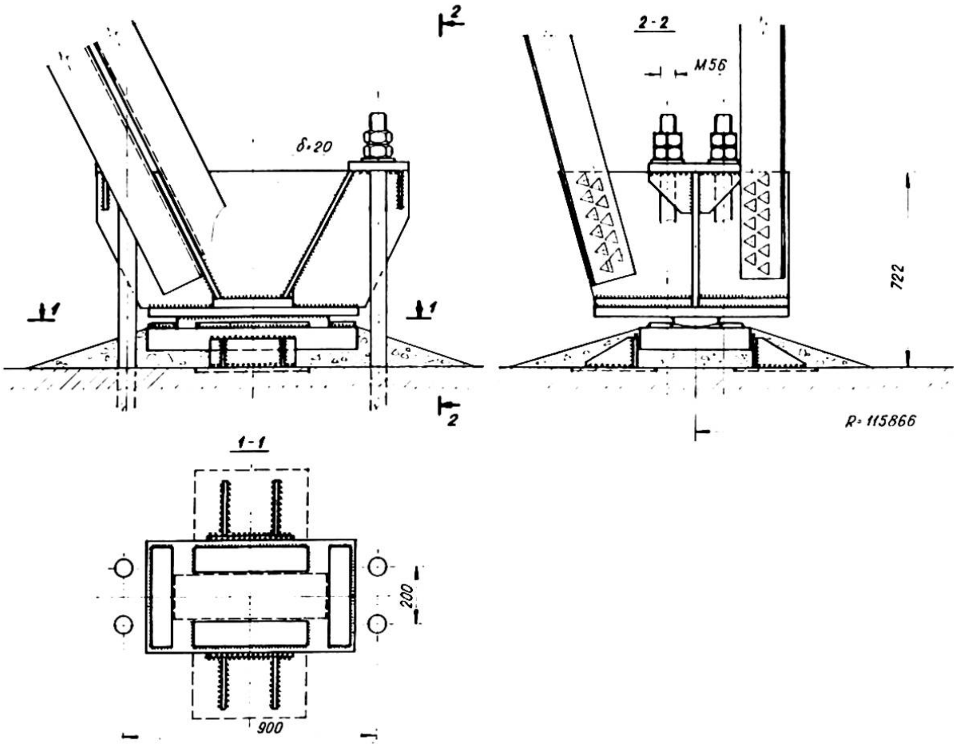


Abb. 5 Auflagerknoten des Gerippes auf das Fundament

im Oberteil und der Hängedecke) beträgt 9829 t (108 kg pro 1 m^2 Fläche), der Aluminiumverbrauch - 363 t (4 kg/m^2).

Die konstruktive Lösung des Traggerippes des Gebäudes in der UdSSR ist mit einem Erfinderzeugnis Nr. 590414 geschützt.

IV

Load-Deflection Characteristics of Tubular Steel Towers

Caractéristiques charge-déformation de pylônes tubulaires en acier

Last- und Durchbiegungs-Eigenschaften von Stahlrohrmasten

NOBUHIRO SATOH
Dr., Project Manager
Tomoegumi Iron Works, Ltd.
Tokyo, Japan

TOSHIRO SUZUKI
Dr., Associate Professor
Tokyo Institute of Technology
Tokyo, Japan

SUMMARY
A design concept for tubular steel electrical transmission towers which are required to be earthquake and wind resistant is described. Experimental and non-linear analyses are performed to investigate the load-deflection characteristics and ultimate strength of the towers, designed in accordance with this concept.

RESUME
Une conception de pylônes tubulaires en acier pour lignes à haute tension devant résister aux séismes et au vent est présentée. Le résultat de l'expérience et l'analyse non-linéaire sont décrits, le but étant de déterminer les caractéristiques charge-déformation et la résistance maximale de l'ouvrage, qui a été réalisée conformément à cette conception.

ZUSAMMENFASSUNG
Ein Entwurfskonzept für Hochspannungsmasten aus Stahlrohren, die gegen Erdbeben und Wind widerstandsfähig sein sollen, wird hier dargestellt. Der Versuch und die nichtlineare Analyse werden beschrieben, um die Last- und Durchbiegungs-Eigenschaften und die Maximalstärke der Masten, die nach diesem Konzept entworfen wurden, zu untersuchen.

1. INTRODUCTION

An electrical power demand is increasing in Japan and the construction of ultra high voltage electrical transmission lines for power supply to big city areas is needed. Self-supporting square towers of double warren trusses are adopted in Japan as the towers of transmission lines, and such ultra high voltage transmission towers will be rigidly jointed trusses using tubular steel members, as shown in Fig. 1. It is very important to keep the integrity of transmission facilities, considering their social influence. In discussing the safety of structures in Japan, the earthquake and wind resistant design is the most important factor. It is necessary to define the load-deflection characteristics and ultimate strength of structures, because the deformation capacities of structures play an important role in the earthquake and wind resistant design.

This study consists of the following items.
- Design philosophy considering the deformation capacities of towers.
- Experiments for determination of load-deflection characteristics up to ultimate state and ultimate strength of towers, designed according to above item.
- Development of a nonlinear analytical method which can predict the characteristics indicated in above item.

2. DESIGN PHILOSOPHY

The slenderness ratios of the tubular members of ultra high voltage electrical transmission towers are expected to be in the range shown in Table 1. The collapse process of the body, the arm and the leg should be such that, assuming the distance between centers of nodes as the buckling length, the bracing members do not buckle before the column members effectively display their strength and deformation capacity in individual buckling. When buckling of column members which have considerably small slenderness ratios as shown in Table 1 precedes, the tower is expected to display deformation capacities because of the column members displaying sufficient stress-carrying capacities with plastic deformations.

Deformation capacity of the whole tower will be increased, if stresses in all members of the tower are uniformly distributed and all panels yield simultaneously. We suggest that the arm should be designed such that it fails earlier. This will contribute to the prevention of a failure of the body, thereby lessening damages and shortening the schedule of restoration works.

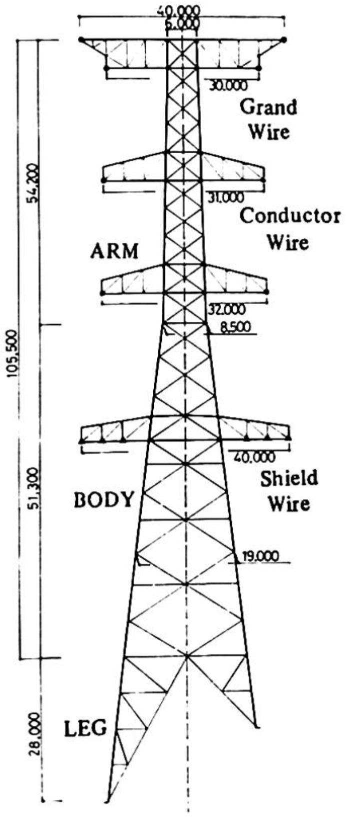


Fig. 1 Example of ultra high voltage electrical transmission towers (mm)

Table 1 Slenderness ratios of members

		Column member	Bracing member
Body	Upper from changed point of inclined angle of columns	30 ~ 60	60 ~ 80
	Lower from changed point of inclined angle of columns	20 ~ 40	80 ~ 150
Arm		30 ~ 40	40 ~ 100
Leg		10 ~ 40	80 ~ 150

3. NONLINEAR ANALYSIS

This analytical method is based on the finite deformation theory derived from the principle of the virtual work. The equilibrium equations of members are formulated for a finite element model in three dimensional stress condition. Regarding heavily deforming or yielding members, we can get higher accuracy of the analysis by using inner nodes in the members.

The relationship of normal stress σ and normal strain ε in materials is determined such as to satisfy the requirement of stub column tests, as shown at M/My=0 in Fig. 2.
Full plastic moment-axial force interaction curve of tubular members is shown in Fig. 3. As for the members subjected to axial forces and bending moments, the relationships of σ and ε are determined like Fig. 2 by finding the full plastic values for normal stress from Fig. 3 and moving the relationship of σ and ε at M/My=0 parallel after yielding, as indicated in Fig. 2.

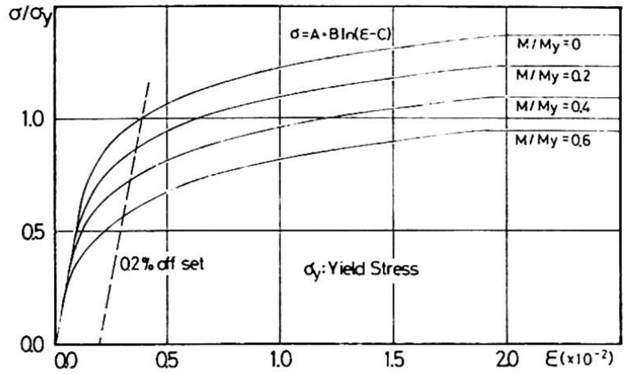


Fig. 2 Stress-strain relationships

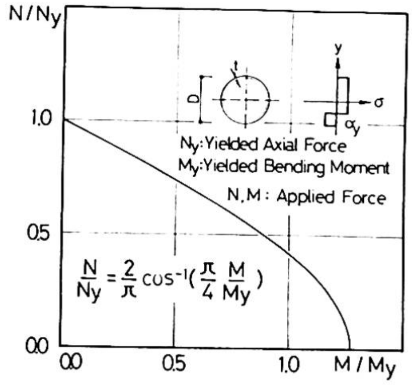


Fig. 3 Moment-axial force interaction curve

4. LOAD-DEFLECTION CHARACTERISTICS AND ULTIMATE STRENGTH OF THE TOWER

4.1 Body

We carried out model experiments on 4 kinds of the body shown in Fig. 4.
The specimens consist of the following factors.
- The specimens are trepaned from the neighborhood of the changed point of the inclined angle of the columns and at the top of the specimens, horizontal forces are caused to act.
- The specimens A and B have different inclined angles of the columns.
  The specimens C and D have the same shape, but different directions of applied forces.
- All the specimens are designed such that buckling of column members precedes that of bracing members. The slenderness ratios of buckled column members are 15 in A and B and 35 in C and D, if the distance between centers of nodes is assuming as the buckling length.
- Column members are continuous and bracing members are connected with column members through gusset plates.

The relationships of load-deflection to force direction given by this experiment and nonlinear analysis are shown in Fig. 5, in which loads given by the conventional analysis, that is to say, the linear static truss analysis with pin-joints of nodes are shown.

The following can be said from the results of the experiment and the analysis.
The yielding load P'y by the nonlinear analysis considering the effect of bending moments corresponds to the load under which the stiffness of the elastic region in the experimental result begins to lower, and the yielding load Py by the

conventional analysis considering axial forces only corresponds to the load under which the stiffness in the experimental result begins to lower rapidly. The ultimate strength P'u by the nonlinear analysis nearly corresponds to the experimental result, but the ultimate strength Pu by the conventional analysis is a little lower than the experimental result. The ductility factors, i.e., the ratios of the deflection at the ultimate strength in the experiment to the deflection at the yielding load by the conventional analysis, are about 3.5 in the specimen A, 2.5 in B and C, and 3.0 in D. The result of the non-linear analysis developed in this study agrees with the experimental result and the availability of this analytical method has been proved.

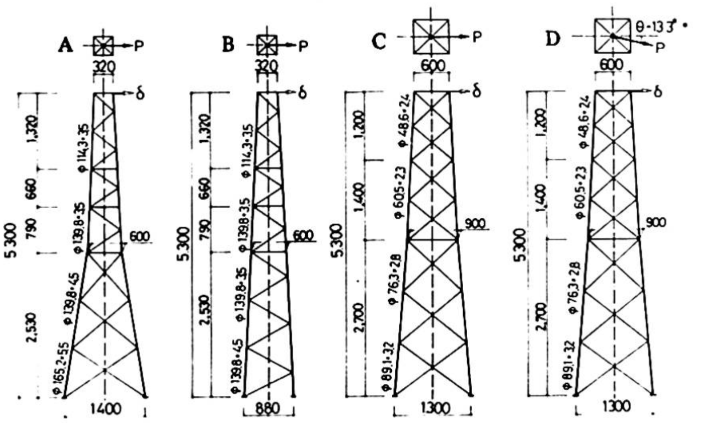
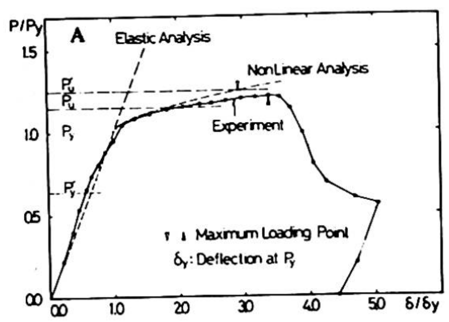


Fig. 4 Specimen of body (mm)



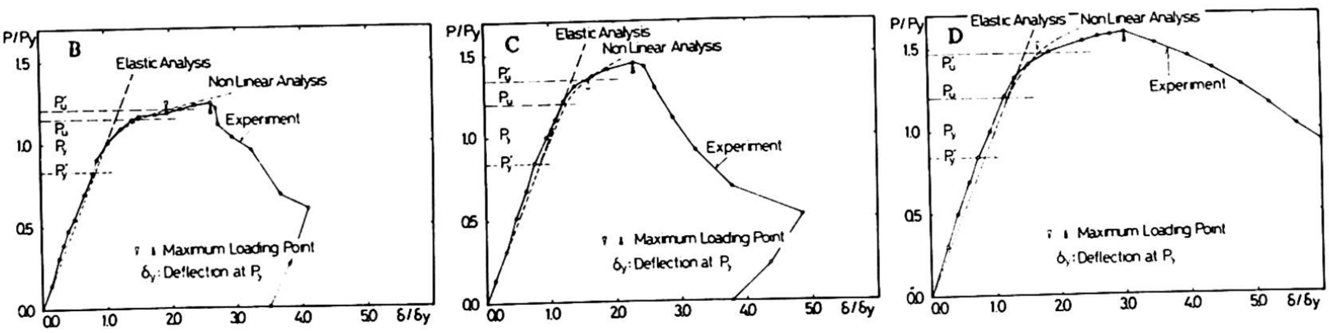


Fig. 5 Load-deflection relationships in body

4.2 Arm

Regarding 3 kinds of the arm shown in Fig. 6, full scale field experiments were performed. The relationships of load-deflection given by the experiment and the nonlinear analysis in the specimen C are shown in Fig. 7. We can obtain almost the same result on the arm as on the body.

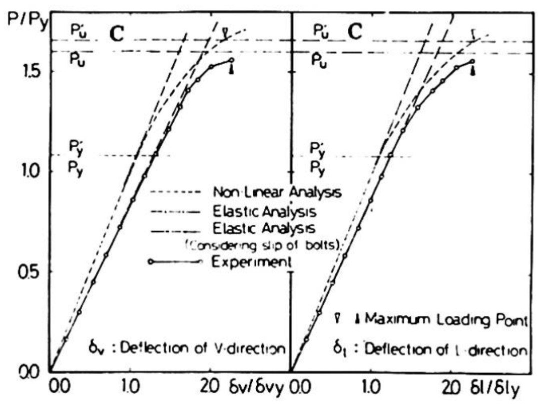


Fig. 7 Load-deflection relationships in arm

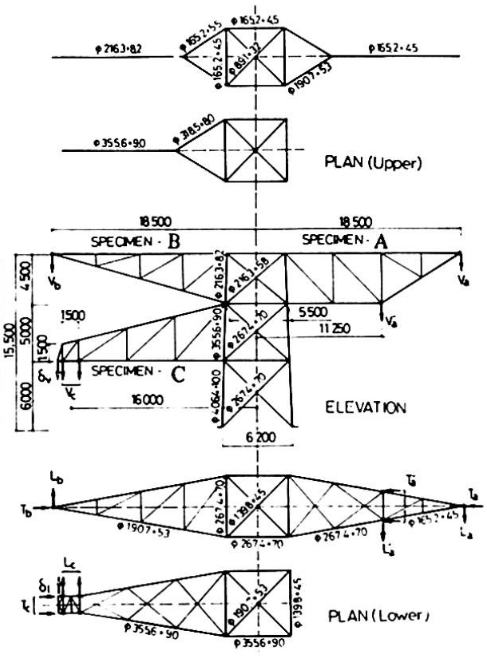


Fig. 6 Specimens of arm (mm)

4.3 Leg

Model experiments were performed on 8 kinds of the leg shown in Fig. 8 and Table 2. The specimens consist of the following factors.

- Variable factors of the specimens are slenderness ratios of column members, number of panels and strength ratios of column and bracing members, as shown in Table 2.
- The ratio of applied forces between column members and bracing members is 10.
- All the specimens are designed such that yielding and buckling of column members precede those of bracing members, except the specimen G.
- Column and bracing members are continuous and reinforcement members are connected with them through gusset plates.

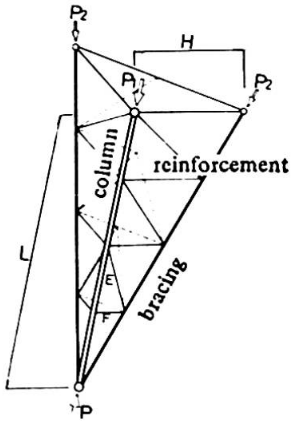


Fig. 8 Specimens of leg

Prediction of buckling load of the leg

The relationships of load-deflection are found by the nonlinear analysis described in chapter 3. Regarding

Table 2 Specimens and results of experiment and analysis

Specimens	Slenderness ratios of column members	Numbers of panels	Ratios of allowable loads: column / bracing	e-member / column	f-member / column	Buckling load: experiment / theoretical	Failed member
A	20	2	6.3	0.07	0.06	0.92	column
B	20	3	6.0	0.07	0.07	0.90	column
C	20	4	5.9	0.13	0.07	1.03	column
D	20	4	4.9	0.13	0.09	0.99	column
E	38	2	7.6	0.10	0.17	0.96	column
F	38	3	6.1	0.12	0.20	0.91	column
G	38	4	11.3	0.13	0.21	0.97	bracing
H	38	4	5.7	0.13	0.21	0.92	column

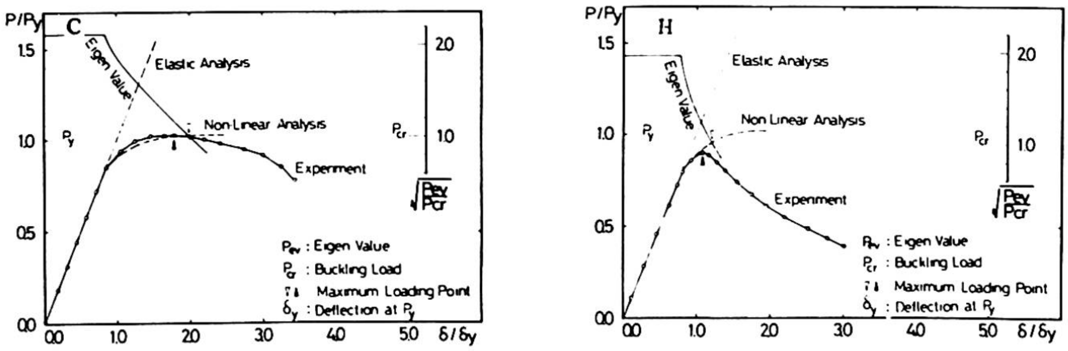


Fig. 9 Prediction of buckling load of leg

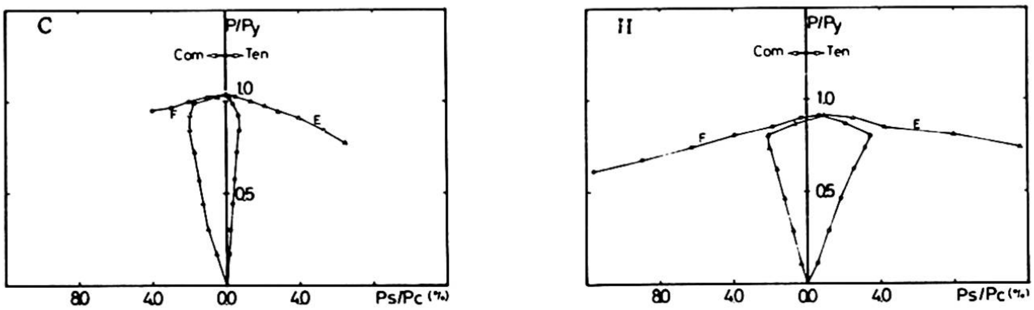


Fig. 10 P/Py–Ps/Pc relationships

each load step in the nonlinear analysis, the eigen values are obtained by performing an eigen value analysis considering material and geometrical non-linearities. In this case, as shown in Fig. 9, we define the load under which those curves intersect as the buckling load. Comparison between the analytical result and the experimental result is made in Table 2. We can estimate the buckling load accurately by using this analytical method.

Necessary strength of bracing and reinforcement members for individual buckling of column members

For the leg design, a method is available in which bracing and reinforcement members are designed as supporting members of the columns. In this method, it is a problem to decide the necessary strength of the supporting members for individual buckling of the column members to be caused firstly in the leg. As regards the specimens C, H, in which column members suffered individual buckling, the ratios of axial forces of the column members Pc to the axial forces of the reinforcement members Ps are shown in Fig. 10. In this experiment, it is obvious that when the strength of bracing members is appropriately overestimated and the strength of the reinforcement members is designed to be about 7% of the strength of the column members, the leg will be failed by individual buckling of the column members.

5. CONCLUSION

Load-deflection characteristics and ultimate strength of tubular steel rigid-jointed truss towers with small slenderness ratios of column members are quantitatively determined in this study. These characteristics will be useful in the earthquake and wind resistant design of electrical transmission line towers.

ACKNOWLEDGMENTS

This study is consigned by the Tokyo Electric Power Company (TEPCO). We heartily wish to thank TEPCO people concerned who gave us various advices and offered kind help.

REFERENCES

[1] R.C. Hensley and J.J. Azar, "Computer Analysis of Nonlinear Truss-Structures", Journal of the Structural Division, Proceeding of the A.S.C.E., June, 1968.

[2] W.S. LaPay and G.G. Goble, "Optimum Design of Trusses for Ultimate Loads", Journal of the Structural Division, Proceedings of the A.S.C.E., January, 1971.

[3] A.K. Noor, "Nonlinear Analysis of Space Trusses", Journal of the Structural Division, A.S.C.E., March, 1974.

[4] T. Suzuki, G. Kawamura, H. Yamagishi, N. Satoh and Y. Takeshima, "Ultimate Strength of Bodies of Electrical Transmission Steel Towers", Summaries of Technical Papers at 1979 Annual Meeting of Archit. Inst. Japan.

[5] T. Suzuki, H. Yamagishi, S. Takao, N. Satoh and K. Izawa, "Ultimate Strength of Arms of Electrical Transmission Steel Towers", Summaries of Technical Papers at 1979 Annual Meeting of Archit. Inst. Japan.

[6] T. Suzuki, T. Ogawa, H. Yamagishi and T. Hiroki, "Ultimate Strength of Legs of Electrical Transmission Steel Towers", Summaries of Technical Papers at 1979 Annual Meeting of Archit. Inst. Japan.

[7] T. Suzuki, T. Ogawa, "Buckling Analysis of Reticulated Cylindrical Shell Roofs (Nonlinear Buckling Behavior Rigidly Jointed Truss Shell)", Transactions of Architectural Institute of Japan, February, 1980.

IV

Drei Wassertürme für Riyadh

Three Water-Towers in Riyadh

Trois châteaux d'eau à Riyadh

KUNO BOLL
Dipl. -Ing.
Boll und Partner, Beratende Ingenieure VBI
Stuttgart, BRD

ZUSAMMENFASSUNG
Für die Ausführung von drei fast gleichen ca. 50 m hohen Wassertürmen in Riyadh stand ein modernes Fertigteilwerk, aber nur angelerntes Baustellenpersonal zur Verfügung. Die Kopfbehälter wurden deshalb in Einzelteilen vorgefertigt, an den Turmfüssen zusammengebaut, gedichtet und dann hydraulisch hochgezogen und in den Turmkopfplatten verankert. Die Montage- und Hubvorgänge werden beschrieben.

SUMMARY
For the construction of three nearly identical and approx. 50 m high water-towers in Riyadh a modern prefabrication plant but only semi-skilled labourers on site were available. For this reason, the elevated tanks were prefabricated in sections, assembled at the bottom of the shafts, the waterproofing applied, the tanks lifted hydraulically into their final positions and the lifting tables anchored in the capping slabs of the shafts. The assemblage and lifting operations are described.

RESUME
L'infrastructure disponible pour la construction à Riyadh de trois châteaux d'eau, à peu près identiques et d'une hauteur d'environ 50 m, était représentée par une usine moderne de préfabrication mais par un personnel de chantier uniquement semi-qualifié. Les réservoirs furent constitués, pour cette raison, d'éléments préfabriqués, assemblés au pied des tours, revêtus de leur étanchéité, puis levés hydrauliquement et ancrés dans les dalles de tête des tours. Les opérations de montage et de levage sont décrites ici.

DREI WASSERTÜRME FÜR RIYADH

Für die Wasserversorgung dreier Neubaugebiete gehobenen Wohnungsstandards waren Wassertürme von je ca. 50 m Höhe nötig (Bild 1). Sie stehen exponiert in der Stadtsilhouette von Riyadh. Es war deshalb wünschenswert, sie mit den vorhandenen Mitteln und Möglichkeiten nicht als Allerwelts-Türme zu gestalten. Da die Wohnungsbauten wegen kurzer Termine weitgehend vorgefertigt und montiert worden waren, stand einerseits ein modernes Fertigteilwerk mit qualifizierten Facharbeitern zur Verfügung, andererseits aber nur angelerntes Baustellenpersonal.

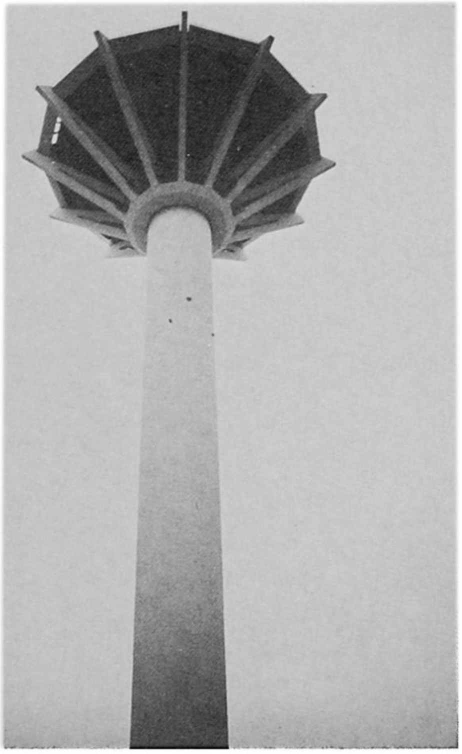


Bild 1 Einer von drei Türmen

Diese Gegebenheiten forderten kategorisch, die Türme - zumal es sich um drei fast gleiche handelte - weitestmöglich vorzufertigen. Die als Generalübernehmerin ausführende Firma Pegel & Sohn erkannte die Vorteile des Einsatzes hochwertiger Technik, sogar zu Lasten eines Mehrverbrauchs an Material.

Funktionell gefordert war unbedingte Dichtigkeit nicht nur des Hochbehälters, sondern auch des Grundtanks, der auf nahezu ganze Höhe auftriebsgefährdet im verseuchten Grundwasser steht, sowie eine Abdeckung des Hochbehälters, um durch die Sonne das Wasser nicht zu heiß werden zu lassen.

Für die Formgebung der Hochbehälter stand alte saudische Architektur Pate (Bild 2).

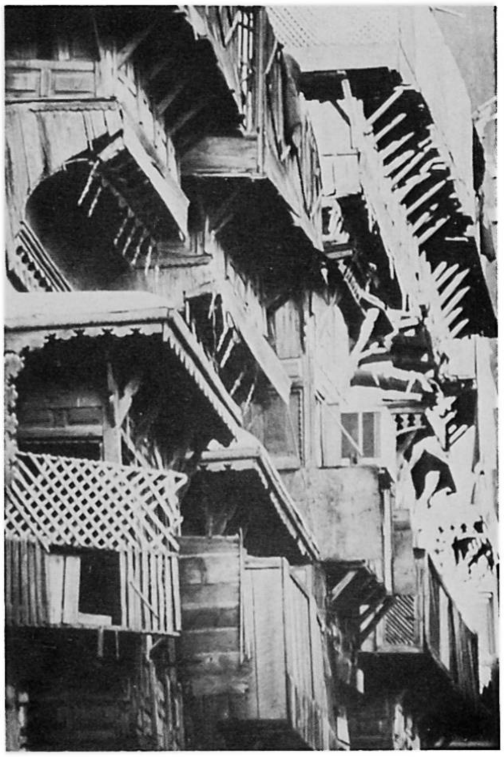


Bild 2 Alte Häuser in Jeddah

Unter Würdigung all dieser Umstände schlugen wir dem Unternehmer vor, zunächst das Turmfundament und darauf den Turmschaft in Gleitschalung und anschließend den Grundtank am Ort zu betonieren. Eine Fertigungsstätte für Schleuderbetonrohre war nicht vorhanden, sonst hätten auch die Schäfte montiert werden können. Der Hochbehälter sollte jedoch in seinen Einzelteilen im Werk vorgefertigt, antransportiert, auf der Grundtankdecke zusammengebaut und bereits mit einer Innenauskleidung versehen hydraulisch gehoben und an der Kopfplatte des Turmschaftes endgültig verankert werden. Der Vorschlag wurde akzeptiert und der Bauausführung zugrunde gelegt.

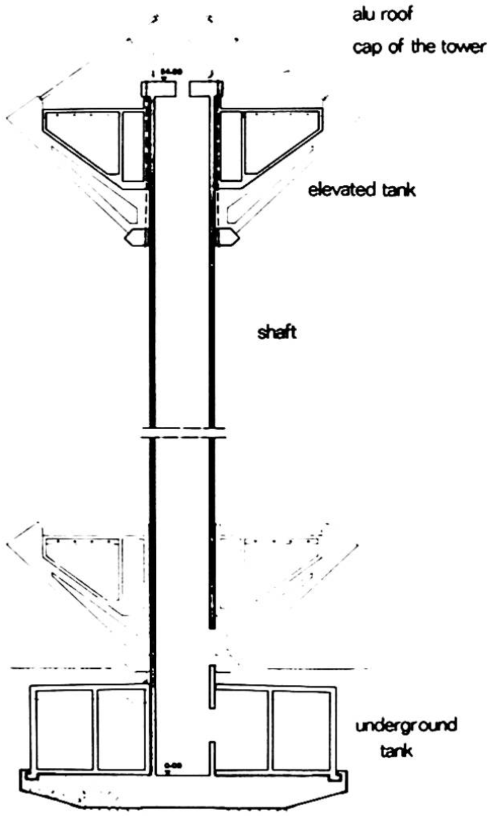


Bild 3 Gesamtschnitt

Der Turmfuß (Bild 4)

Der felsartige Baugrund erlaubte eine kreisringförmige Gründung; die Fundamentplatte wurde deshalb unter dem Schaft und unter den äußeren abgeschrägten Bereichen, auf denen die Außenwand des Grundtanks steht, auf Polystyrolplatten weich gelagert. Der Grundtank selbst wurde zwängungsfrei, d. h. horizontal verschieblich, auf die Fundamentplatte aufgesetzt. Gegen Auftrieb schützt ihn außen eine Verzahnung in der Fundamentplatte, im Schaftbereich eine Verankerung mit Hilfe von Gewi-Muffen-Stäben. Die Tankdecke wurde auf 4 cm dicke vorgefertigte Stahlbeton-Schalungsplatten 21 cm hoch aufbetoniert. Sie stützt sich auf vorgefertigte Stahlbetonträger, die auf den Tankwänden aufliegen.

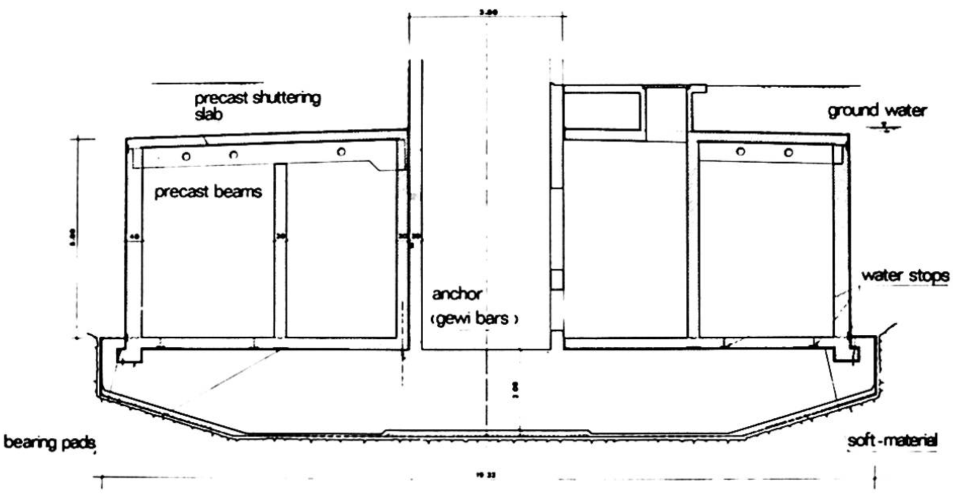


Bild 4 Schnitt durch das Turmfundament und den Bodentank

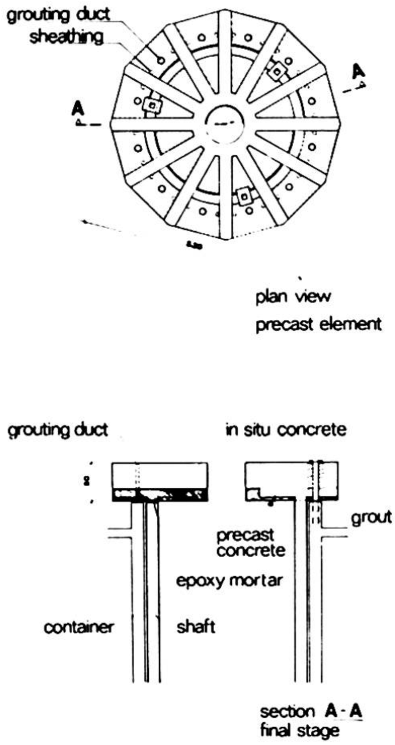


Bild 5 Teilvorgefertigte Schaftkopfplatte

Der Turmkopf (Bild 5)

Die Kopfplatte des Turmschaftes wurde im Werk 8 cm dick mit 30 cm hohen radialen Rippen teilvorgefertigt, bemessen für die Last von 82 cm Ortbeton. Die gesamte Bewehrung einschließlich der Spanngliedverankerungen, der Wendeln und aller Aussparungen wurde im

Werk eingebaut, so daß nach dem Versetzen mit Hilfe des Krans nur noch der Ortbeton innerhalb der am Fertigteil befestigten Stirnschalungen eingebracht werden mußte.

Der Hochbehälter hat ein Gesamtgewicht von ca. 8800 KN (880 t) (Bild 6). Seine Konstruktion besteht aus:

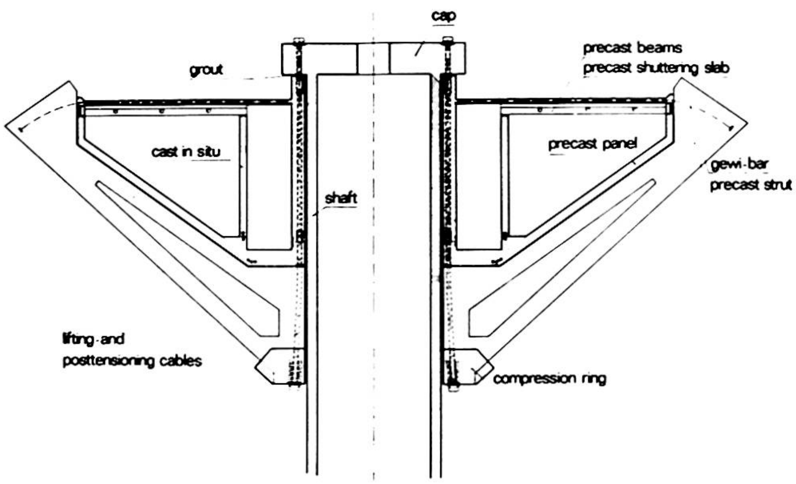


Bild 6 Schnitt durch den Hochbehälter

1.) Dem Haupttragwerk, dessen bemerkenswerteste Bestandteile die 12 vorgefertigten zweiarmigen Schrägstreben sind (Bild 7). Sie sind im Endzustand vertikal gehalten durch die im Strebenfuß verankerten Spannglieder. Horizontal sind sie im unteren Bereich durch den massiven Ortbetondruckring und die Ortbetonbodenplatte des Innenbehälters gestützt. Die Horizontalkräfte am oberen Strebenende werden mit Gewi-Muffen-Stäben in die Kreisringscheibe der Behälterdecke eingeleitet.

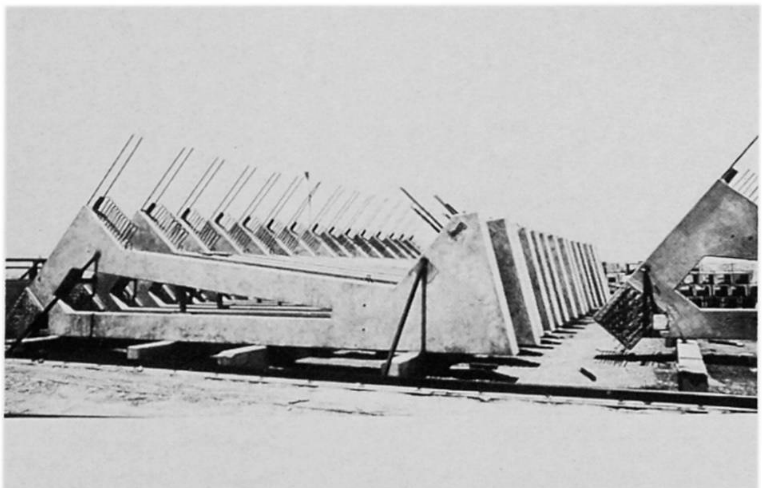


Bild 7 Die Streben sind abholbereit

2.) Dem Nebentragwerk, das die raumabschließenden Teile des Hochbehälters umfaßt. Die äußere Behälterwand - also die Kegelfläche - setzt sich aus vorgefertigten Stahlbetonsegmentplatten zusammen, die auf den Streben aufliegen. Die Streben wirken daher als schrägliegende Auflagerbalken und werden als solche nicht nur gedrückt, sondern auch gebogen. Auf das Tragverhalten einer Kegelschale wurde bewußt verzichtet.

(Bild 8) Zur Herstellung des Hochbehälters auf dem Grundtank wurden zunächst die Streben innen auf vorgefertigten Betonstelzen und außen auf Hilfsrüstungen maßgerecht versetzt; anschließend wurde der Druckring zwischen den Streben betoniert. Auf ihnen wurden die trapezförmigen Platten verlegt, in die die Fugenbänder bereits einbetoniert waren. Letztere wurden nach dem Verlegen sorgfältig verschweißt und dann die dafür notwendigen Aussparungen am Ort vergossen. Dann wurde der Behälterboden betoniert und die vertikalen Behälterwände und schließlich die Behälterdecke am Ort ergänzt. Für letztere wurden wieder vorgefertigte Träger und vorgefertigte großformatige 4 cm dicke Stahlbeton-Schalungsplatten mit Gitterträgern verwendet.

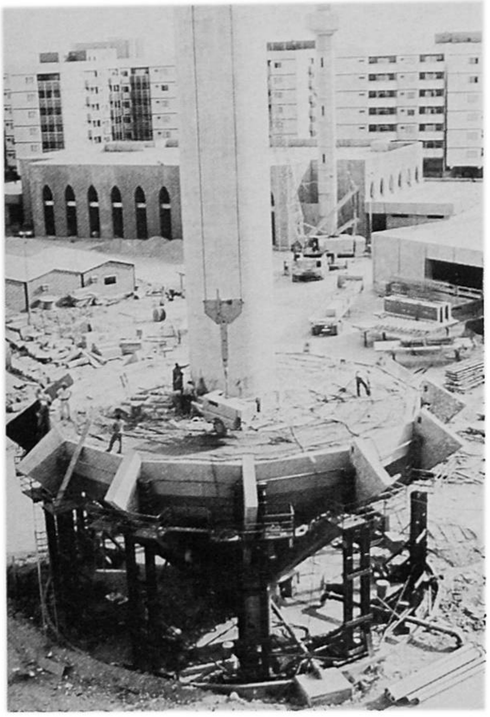


Bild 8 Der Hochbehälter nach Abschluß der Betonierarbeiten

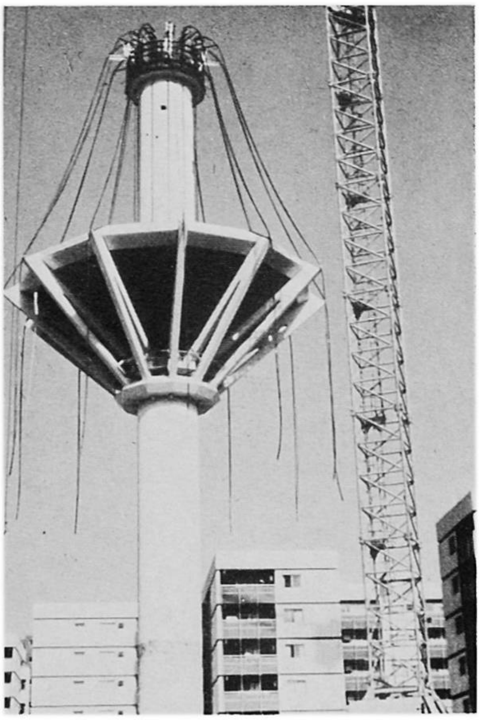
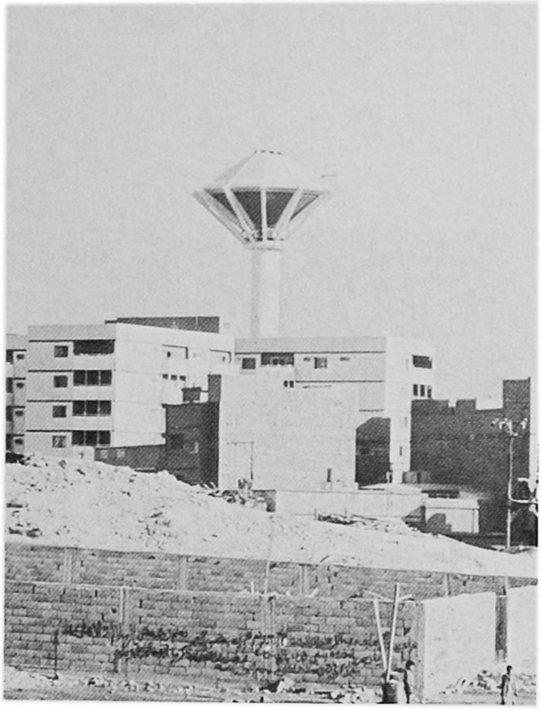


Bild 9 Der Hochbehälter wird gehoben

Als letztes erhielt der Hochbehälter eine Innenauskleidung. Sie mußte wegen der zu erwartenden Bewegungen zwischen den Fugen der Trapezplatten flexibel sein. Gewählt wurde eine glasfaserbewehrte Epoxidharz-Beschichtung auf aufgeklebter Bitumenschweißbahn, die im Bereich der Fugen zur besseren Überbrückung von Bewegungen durch eine Unterlage vom Beton getrennt wurde.

Anschließend wurde der Hochbehälter mit Litzenspanngliedern System Losinger bis unter die Kopfplatte des Turmes hochgezogen (Bild 9); die zu hebende Last betrug ca. 6500 KN (650 t). Gehoben wurde mit 12 auf der Schaftkopfplatte verankerten Spezialhubpressen; die Spannglieder bestanden beim Heben aus je 7 Litzen. Für die endgültige Aufhängung und Vorspannung des Hochbehälters wurden je 3 kürzere Litzen mitgeführt.

Um eine kontinuierliche Kontaktfläche zwischen dem Hochbehälter und der Kopfplatte zu erzielen, wurde der Behälter bis auf ca.2 cm an die Kopfplatte herangezogen. Die Hubgeräte wurden abgebaut und der vorhandene Zwischenraum wurde durch 80 mm PVC-Rohre, die in der Kopfplatte angeordnet waren, mit Fließmörtel verfüllt (vgl. Bild 5). Nachdem dieser abgebunden hatte, wurde litzenweise die Vorspannung aufgebracht und anschließend wurden die Hüllrohre der Spannglieder injiziert.



Die Behälterdecke erhielt gegen Oberflächenwasser eine Dichtung mit Gehbelag. Für den Schutz des Behälters gegen Sonneneinstrahlung dient eine dachfärmige Aluminiumabdeckung. Für Kontroll- und Wartungszwecke ist zwischen den Streben ein Umgang angeordnet.

Die Türme sind fertiggestellt (Bild 10).

Bild 10 Ein fertiger Turm

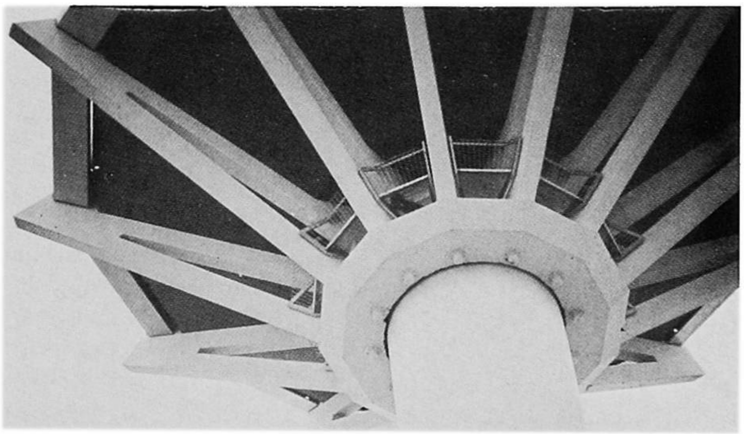


Bild 11 Behälter-Untersicht

IV

Les couvertures en béton léger de l'aérogare No 2, aéroport Charles de Gaulle

Leichtbetondecke des Flughafengebäudes Nr. 2, Flughafen Charles de Gaulle

Lightweight Concrete Roof of the Air Terminal No 2, Charles de Gaulle Airport

BERNARD RASPAUD
Ingénieur
Entreprise Bouygues Travaux Publics
Clamart, France

RESUME
Les caissons de couverture de l'aérogare no 2 de l'aéroport Charles de Gaulle (structures en béton léger précontraint de 40 m de portée) sont exceptionnels:

1. par leur mode de réalisation: préfabriqués au niveau plancher, ces caissons de 1 200 t sont hissés à leur niveau définitif par un système de vérins hydrauliques
2. par le volume de béton léger utilisé: 15 000 m^3 ont été nécessaires (chiffre très important, comparé aux autres réalisations).

ZUSAMMENFASSUNG
Die Caissonelemente für die Decke des Gebäudes Nr. 2 des Flughafens Charles de Gaulle (Struktur aus Leicht-Spannbeton mit einer Tragweite von 40 Meter) sind ausserordentlich:

1. wegen ihrer Bauart: vorgefertigt auf Bodenebene werden diese 1 200 Tonnen schweren Caissonelemente auf ihre endgültige Höhe mit einem Hubspindelsystem gehisst
2. wegen des gebrauchten Leichtbetonvolumens: 15 000 m^3 waren dazu nötig (sehr hohe Zahl im Vergleich zu anderen Gebäuden).

SUMMARY
The roof elements of air terminal no 2 of the Charles de Gaulle airport (structures in lightweight prestressed concrete of 40 m span) are quite remarkable:

1. by the way in which they are constructed: these elements of 1 200 tons each are precast and then elevated to their final position
2. by the volume of lightweight concrete used: 15 000 m^3 ; a considerable figure in comparison with previous uses of this product.

En mai 1978, l'Aéroport de Paris a désigné notre Société comme entreprise générale pour la réalisation du marché de gros œuvre comprenant les ouvrages d'art des viaducs d'accès, le parc de stationnement des voitures, les modules de trafic A et B, les modules d'entrée I et J et les modules de jonction K et L. Cet ensemble constitue le premier des quatre anneaux prévus pour l'aérogare n° 2.

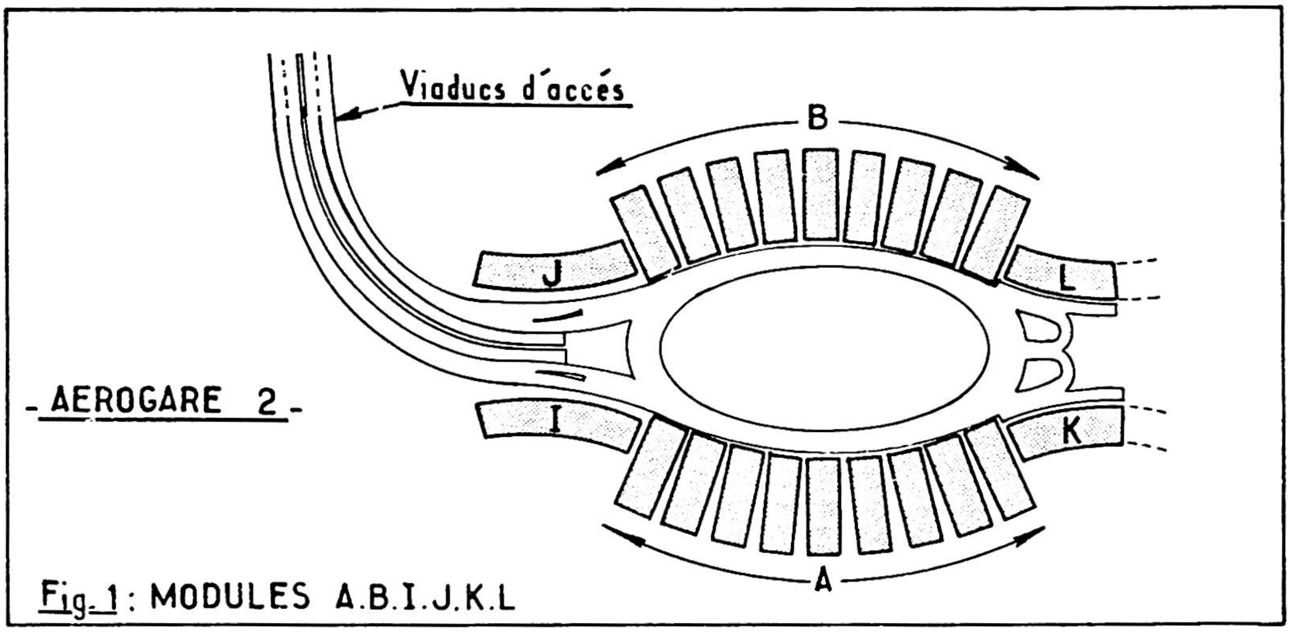
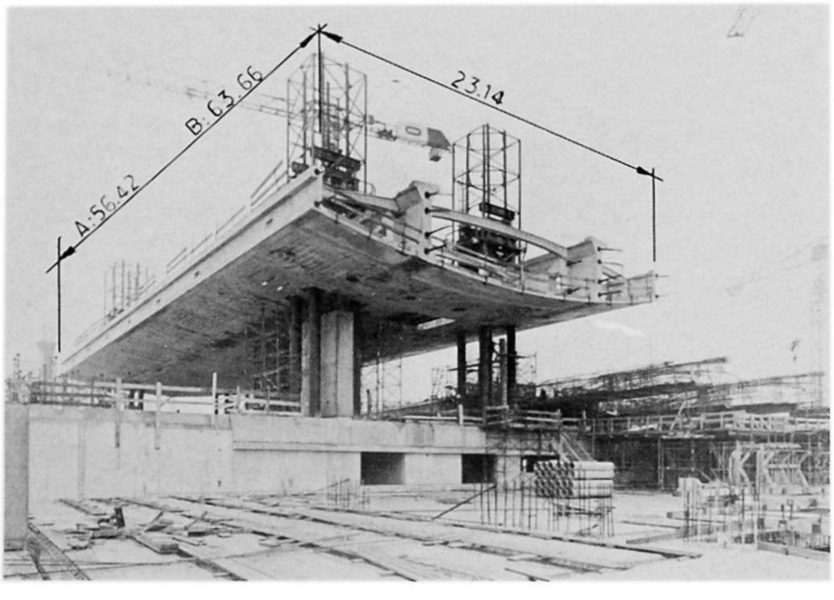


Fig. 1 : MODULES A.B.I.J.K.L

Les structures de ces ouvrages sont, dans l'ensemble, assez conventionnelles en dehors toutefois des couvertures des modules A et B.

Ces couvertures sont constituées de 9 caissons de 23.14 m de largeur dont la longueur est de 56.42 m pour le module B et de 63.66 m pour le module A. Chaque caisson repose sur 4 poteaux espacés de 11.90 m dans le sens transversal et de 40.30 m dans le sens longitudinal.



Les éléments porteurs essentiels de cette structure en béton léger précontraint sont les deux poutres principales de 3.44 m de hauteur dont l'épaisseur varie de 0.40 m en travée à 0.80 m sur appui. Ces deux poutres parallèles sont distantes de 11.90 m d'axe en axe.

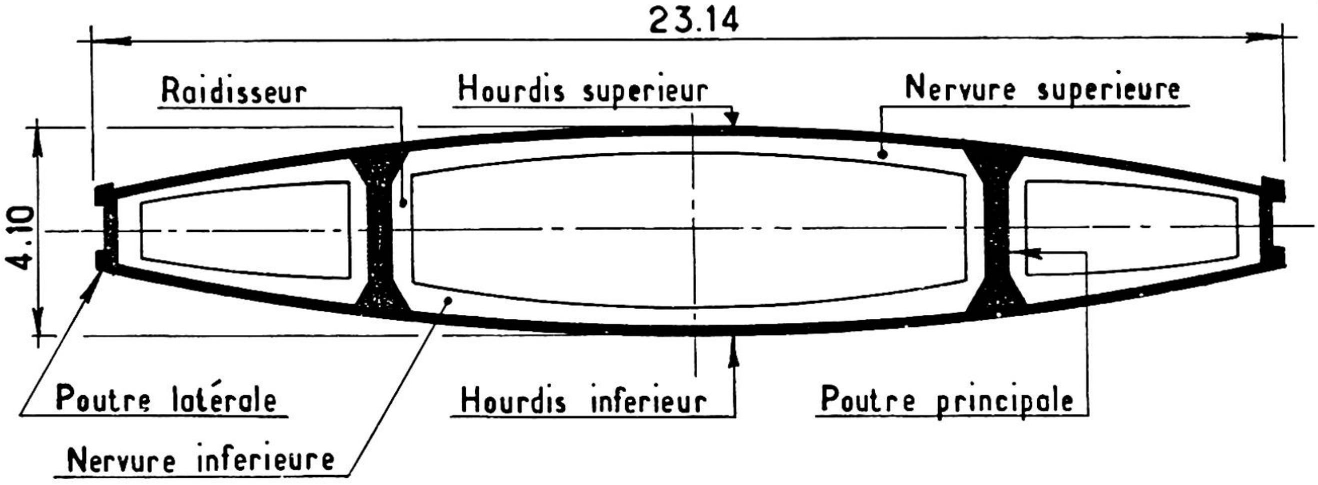


Fig. 2: MODULE A : longueur 63.66 m
MODULE B : longueur 56.42 m

A l'extérieur des poutres principales et parallèles à celles-ci, les deux poutres latérales limitent le caisson. Leur hauteur est de 1.60 m et leur largeur de 0.20 m.

Dans le sens transversal, des nervures en béton armé de 0.50 par 0.30 m ont été disposées tous les 3.62. Ces nervures règnent en partie supérieure et en partie inférieure du caisson sur toute la largeur ; les nervures supérieures sont réunies aux nervures inférieures par des raidisseurs verticaux disposés sur les poutres principales et latérales.

Ces nervures supportent des dalles de 0.12 m d'épaisseur formant les hourdis supérieur et inférieur.

L'étude de ces caissons nous a conduit à envisager le schéma statique suivant :

- les poutres principales et le hourdis qui leur est associé représentant 97 % de l'inertie totale de flexion reprennent la quasi totalité des charges.
- les nervures transversales reprennent bien sûr les charges amenées par les hourdis mais aussi les charges supportées par les poutres de rives qui se comportent comme des éléments suspendus aux nervures transversales. Ces nervures transmettent les charges aux poutres principales.

L'étude de cette structure n'a pas présenté de difficultés particulières en dehors des problèmes liés à la diffusion de précontrainte qui ne pouvaient être traités par une simple application des règles classiques et qui ont conduit à renforcer le ferraillage dans les zones d'extrémité.

Dès le début, les problèmes essentiels ont consisté pour nous, à mettre au point le système de levage des caissons d'une part, et le béton léger d'autre part.

Le béton léger devait avoir une résistance nominale à la compression de 300 bars. Après avis du SETRA (qui collabora avec l'Aéroport de Paris et nous-mêmes à la résolution des problèmes que posait ce matériau), il fut décidé en accord avec l'Aéroport de Paris, d'adopter une densité de calcul de 1.8 t/m^3 (au lieu de 2.5 t/m^3 pour le béton ordinaire).

En raison des quantités nécessaires (15 000 m^3 de béton léger) une seule source ne pouvait assurer la production de granulats. Nous utilisons donc trois origines différentes : des 4/10 fournis par les Granulats Expansés de la Mayenne, des Surrex 4/10 et des Surrex 6/12. Le ciment est du CPA 55 et le dosage en eau est de 180 litres par mètre cube, plus 40 litres pour le mouillage des agrégats.

La résistance nominale de ce béton est toujours supérieure à 350 bars et souvent voisine de 400 bars, donc bien supérieure aux 300 bars exigés ; par contre, la densité de calcul atteint 1.83 t/m^3 au lieu de 1.8 visés au départ.

Ce béton dont le coût est supérieur de 30 % au béton ordinaire ne nécessite pas de dispositions particulières lors de sa mise en œuvre, et, avec un traitement thermique, la résistance atteint 120 bars lors du décoffrage à 16 h.

Le caisson est préfabriqué au niveau du dernier plancher à l'aide d'un coffrage permettant de couler chaque jour une « tranche » de 3.62 m de caisson. D'abord le hourdis inférieur et les nervures inférieures, puis les poutres principales et latérales, et enfin les nervures supérieures.

C'est après précontrainte et avant coulage du hourdis supérieur que le caisson est hissé à son niveau définitif, le bétonnage de la dalle supérieure n'intervenant que lorsque le caisson repose sur ses appuis définitifs.

Le système de levage permet de hisser de 6.00 m environ le caisson de 1 200 tonnes. Pour cela, nous utilisons quatre palées métalliques, composées chacune de quatre tubes de 0.61 m de diamètre et de 16.4 mm d'épaisseur. La longueur de ces tubes est de 16 m, ils traversent le plancher sur lequel les caissons sont préfabriqués et reposent sur les fondations de l'ouvrage.

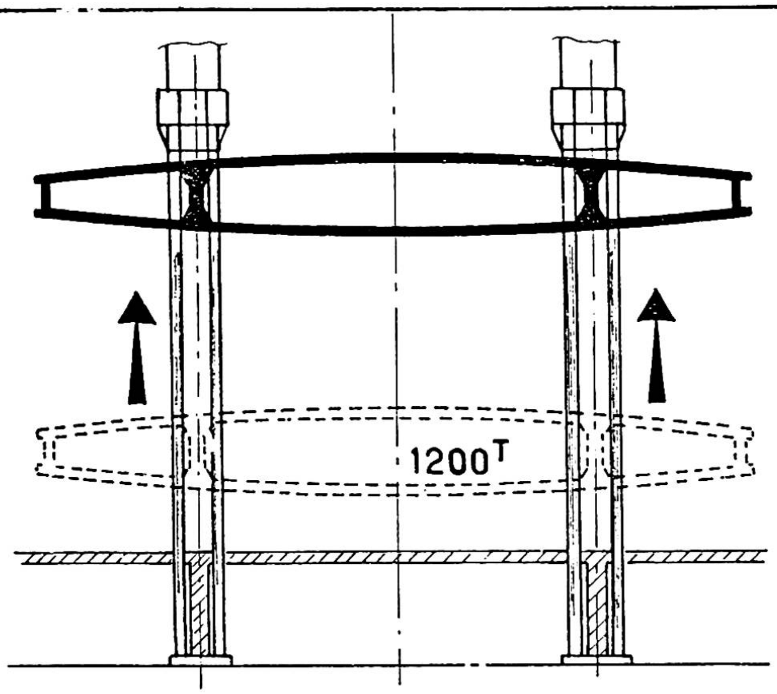


Fig 3: COUPE TRANSVERSALE

Le levage est assuré par deux batteries de 16 vérins de 200 t travaillant alternativement avec reprise des efforts d'une batterie à l'autre par l'intermédiaire de deux chevêtres mobiles brochés aux suspentes.

Sur une même palée, les 4 vérins restent toujours en liaison hydraulique mais chaque palée est indépendante. Il faut donc veiller à ne pas créer de torsion inadmissible dans le caisson. La dalle supérieure n'étant pas encore coulée, la souplesse en torsion de celui-ci permet de déniveler un des appuis de 4 cm par rapport au plan défini par les trois autres. Le contrôle de cette torsion est assuré pendant le levage par un système de niveau d'eau aux quatre coins du caisson avec renvoi et contrôle visuel au centre du caisson, à côté de la centrale de commande des vérins. Ce contrôle ne pose pas de problème particulier et dès le 3ème caisson le temps de levage est descendu à 6 heures environ.

Pendant le levage, le caisson étant accroché à l'extrémité des suspentes peut se balancer librement. Sous l'action des rafales de vent on risquait d'observer des résonnances entre la période des tubes constituant les palées et la période du caisson se balançant à l'extrémité des suspentes.

Pour éliminer tout danger et travailler avec les coefficients de sécurité normaux, nous avons été amenés à prescrire un arrimage des caissons contre les poteaux dès que le vent dépasse 40 km/h, la station météorologique de l'Aéroport nous prévenant 2 heures à l'avance si des vents importants sont annoncés.

Ces dispositions ont permis d'assurer un réemploi satisfaisant de la charpente de levage pour l'ensemble des 18 caissons, celle-ci s'étant toujours bien comportée y compris sous des vents de 130 km/h.

La réalisation de ces caissons qui couvrent une surface de 25 000 m^2 et ont nécessité 15 000 m^3 de béton léger, apportent la preuve d'une maitrise parfaite de ce matériau utilisé ici à une échelle nouvelle et ceci permet d'affirmer qu'il n'y a plus d'obstacle aujourd'hui à de nouveaux développements du béton léger.

IV

Two Concrete Folded Structures for Large Storage Coverings

Deux types de structure plissée pour des grandes toitures

Zwei vorfabrizierte Spannbeton-Faltwerke für Lagerhallen mit grossen Spannweiten

MIRCEA MIHAILESCU
Prof.
Institute of Technology
Cluj-Napoca, Rumania

FELLOW-AUTHORS
Chemical Structural Contractor
Bucharest, Rumania

SUMMARY
The first structure consists of three-hinged arches with a folded structure. The 60 m span is made up of two thin precast and prestressed concrete units. The second structure has a set of fixed-end frames. The 30 m span is made up of two precast and prestressed elements. The bearing capacity was increased by providing the beams with prestressed tie bars. In both situations, asbestos-cement sheetings were placed between the folded profiles.

RESUME
La structure de la première toiture est constituée d'arcs à trois articulations avec section transversale plissée. La portée de 60 m est franchie par deux éléments préfabriqués en béton armé. La deuxième structure est constituée de portiques de 30 m de portée, ayant également une section transversale ouverte. Les deux portiques ont été renforcés par des haubans précontraints. Dans les deux cas les espaces entre les éléments porteurs ont été couverts avec des panneaux en asbociment.

ZUSAMMENFASSUNG
Im ersten Beispiel besteht die Tragstruktur aus Dreigelenkbogen mit einem dünnen Faltwerkprofil. Die 60 m Stützweite werden von zwei vorfabrizierten Spannbeton-Elementen überspannt. Im zweiten Fall besteht die Hauptstruktur aus eingespannten Rahmen von 30 m Länge, die ebenfalls mit diesem Faltwerkprofil ausgestattet sind. Die Rahmenriegel werden durch vorgespannte Zugstangen verstärkt. In beiden Fällen werden die Räume zwischen der Tragstruktur mit Asbestplatten überdeckt.

1. INTRODUCTION

The field of thin spatial structures is nowadays sensibly enlarged by the applications of shells to the industrial, especially to the chemical, objects, where this class of solutions appears of a high efficiency.

The paper bears on two types of concrete precast and prestressed folded elements, of large spans , designed for chemical depot coverings, in Romania .

The great agressivness of the inside atmosphere was the reason, why the structures had to be situated in the concrete solution field .

2. COVERING ERECTED IN BACAU

The structure in this case essentially consists of transversal three hinged folded arches, crossing a 60m span and reaching a height of 20 m, in order to satisfay the specific functional conditions as Fig.1 shows.

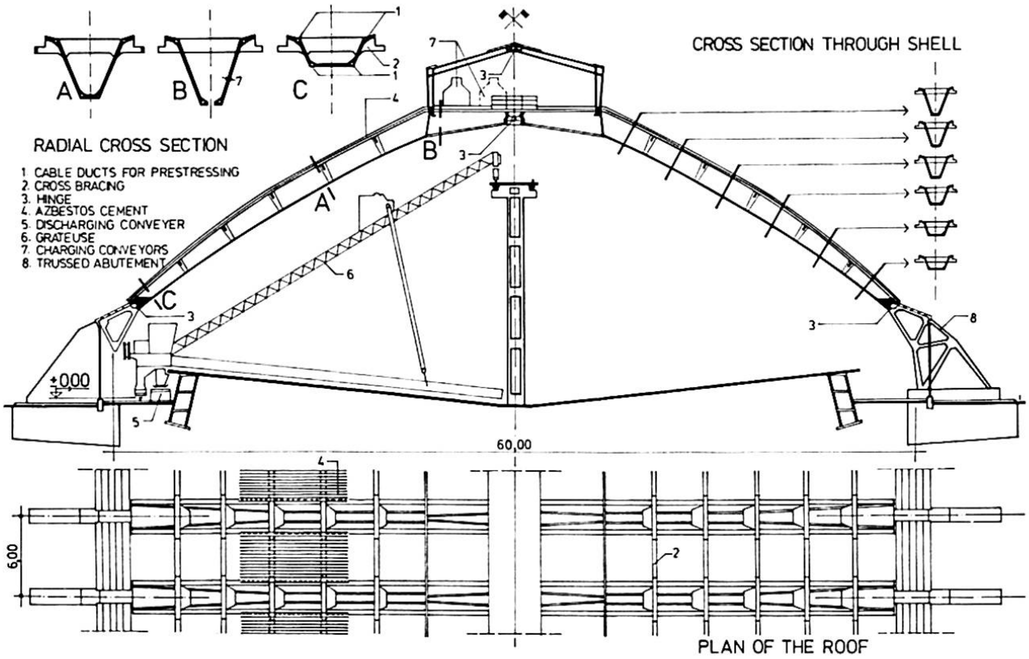
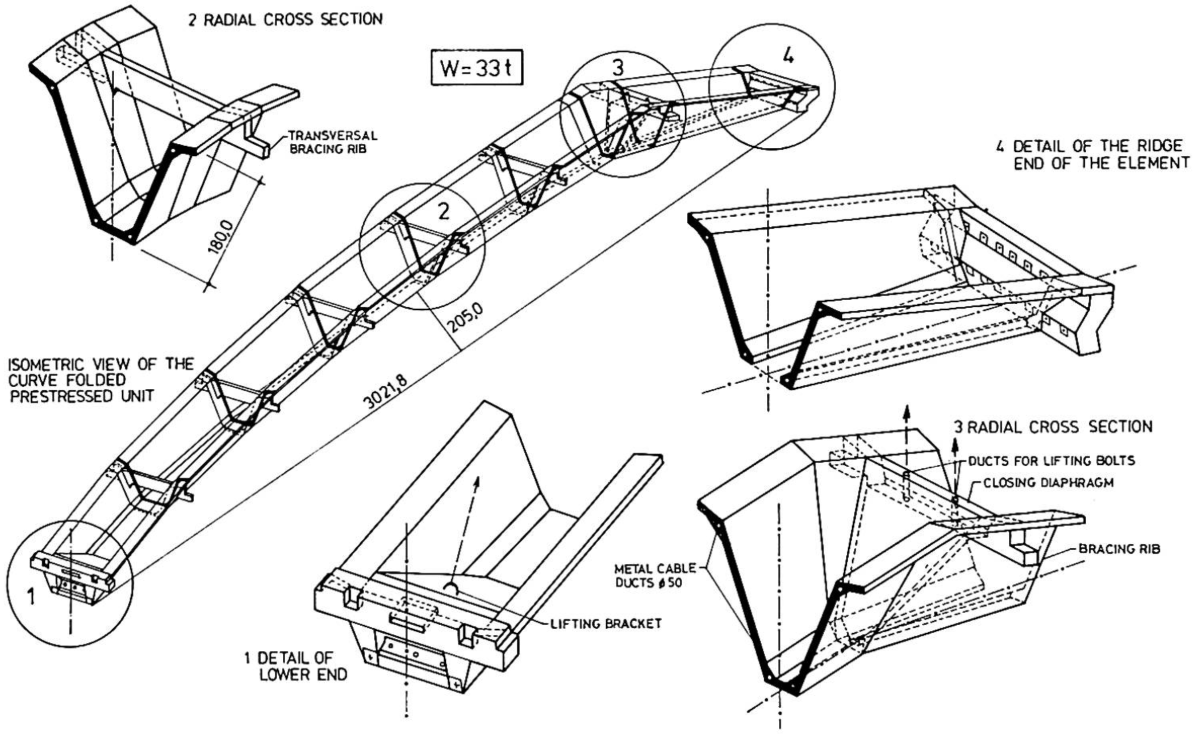


Fig.1. Cross profile of the chemical depot

Fig.2. The longitudinal stiffening system

The arches , each made of two precast thin shell units, of 2.60 m width and variable depth, are placed at 6,00 m longitudinal bay intervals, as seen in Fig.2 ; for their stiffening, in the longitudinal sense bracing ribs and cross bars, were provided, forming two blocks of 85 m each . The intervals between the strips covered by the arches, of 3.40 m, were completed with asbestos cement sheetings and meta-acryl panels, the latter assuring the inside natural lighting , though a 43 percentage economy to the cladding elements was obtained .



Fig, 3. The shape of the concrete precast shell element

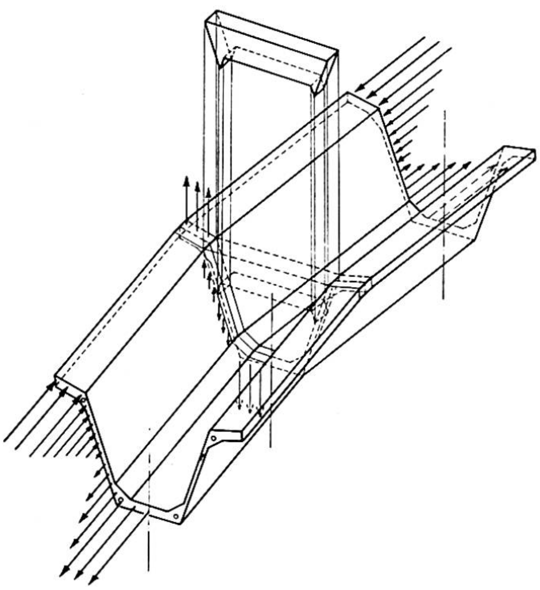


Fig.4.The transversal parasitic efforts .

The curved folded elements of a 32 m axis length, (their fullscale geometry appears in Fig.3) was conceived in a \/ form, so that the centre of the cross section might be located, as near as possible to half of its depth, making the both extreme flanges to be equally solicited, to alternative bending moments .

The shape of the longitudinal axis of the precast elements, was set in an optimal position against the thrust pressure line, carried out on the three hinged system, for the dead load, considered as a short time action .

It is to be mentioned, as an aucommon feature (Fig. 3,4) , that the longitudinal axis was performed, of straight segments of variable slopes and lengths, having in mind to concentrate the parasitic transverse forces , which normallyappear in the flanges of thin curved elements, induced by the axial efforts , in a short number of transition zones, each stiffenned with bracing ribs, so as to reach a box effect.

Among the advantages of the chosen cross profile, there are to point out:

i- a good lateral stability during handling (fig.5) , a big stability to the Brazier effect, especially for negative bending moments, which may frequently occur, a satisfactory local stability , of the flanges and webs of the element, as well as, a great economy of the needed materials and manover.

For each shell element, four types of loading hypothesis within the first order theory, were taken to account : (i) the initial prestress forces of 200 K.N. acting in the horizontal casting position, (ii) the initial prestress forces and dead weight, on the static scheme, which occurs during the handling ,(iii) on the three hinged arch-scheme, the loading combinations, wich separately give the maximum compression efforts in the outside or inside extreme fibers (Fig. 6 shows the first (iii) case).

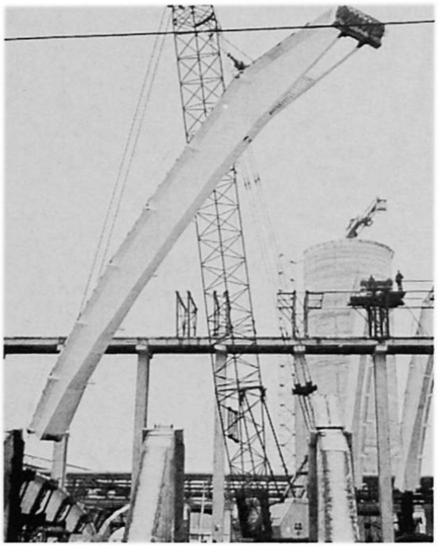


Fig.5. The handling of the precast element

The arch units were reinforced with mild steel bars , and four post-tensioned cables , placed in ducts, with parallel trucks, in the vertical sense, but no longer parallel in the horizontal sense . The bottom hinges Fig.7 made of metal sheets, as to avoid friction against rotation, were placed on metal wedges and finally stiffenned, by welding and in situ poured concrete .

The crown hinges Fig.8 were made of metal cylinders, filled up with concrete, along which, two families of adjustable bolts can rotate,belonging each to one of t he two units .

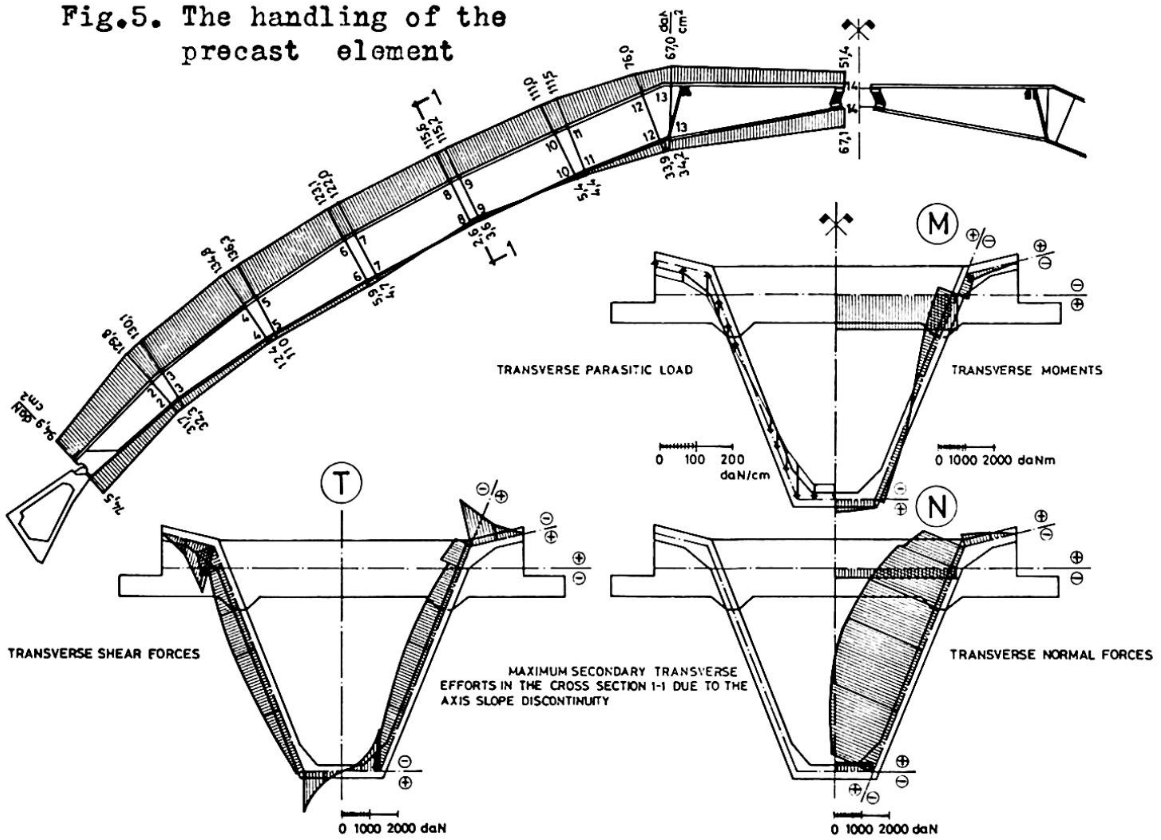


Fig. 6. Maximum compressive stresses at the outside fibres for cumulative actions.

The entire covering matezial indicators were : - concrete Q12 m^3/m^2 and total steel - 18,65 kg/m^2 . The Figures 9 and 10 show outside and inside views ot the finished depot .

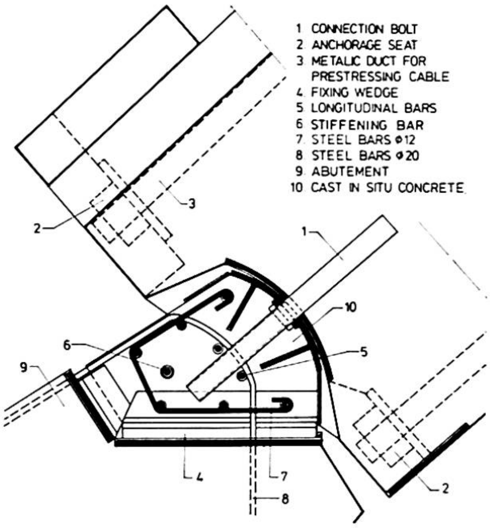


Fig.7 . Bottom hinge

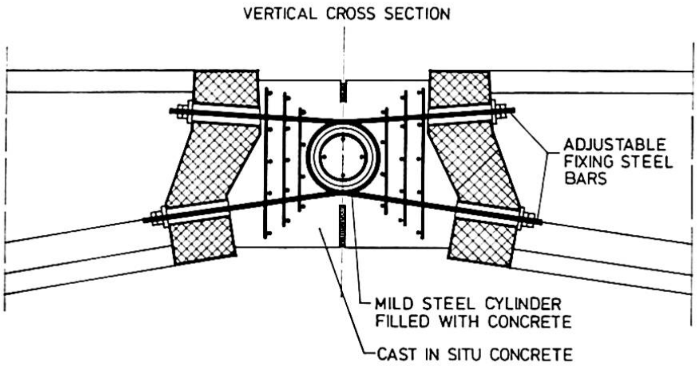


Fig.8. Ridge hinge

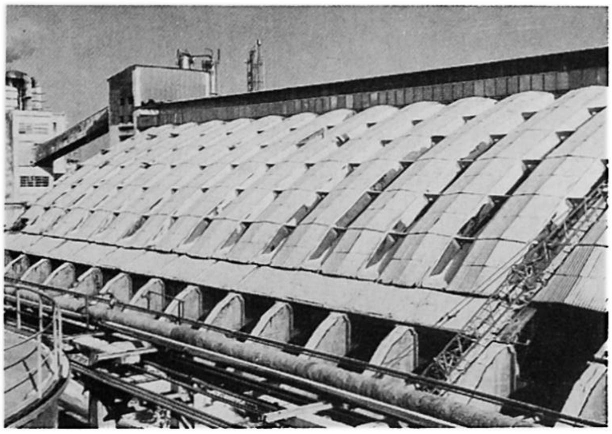


Fig.9. Out side view of the finished depot

Fig.10Inside view of the depot

3. THE COVERING LOCATED IN FAGARAS

The secimd type of chemical depot in Făgăraş , mainly consists of a set of simple fixed frames disposed at 9,00 m bay intervals , each crossing a 30,00 m span , and having a 2.60 m width .

As Fig.11 shows , the geometrical data were chosen, as to satisfay the technological requirements . As mechanical specific loads, there be mentioned two vertical life loads of high values, about lot each, placed at the quarters of the midle span , as well as the horizontal pressure, induced by the stored chemical material, at t he half of the column height .

For efficiency purposes, the frame elements were precast , Fig.12-13, showing the individual pieces. In order to increase the bearing capacity of the horizontal members, the beam effect was supplemented with a tension carrying action, providing a suspended diagonal cable system. Also it was investigated the possibility to minimize, the construction material amount, by conceiving : the columns as truss elements, prestressed at the ouside line, and the beams as thin folded shaped shells.

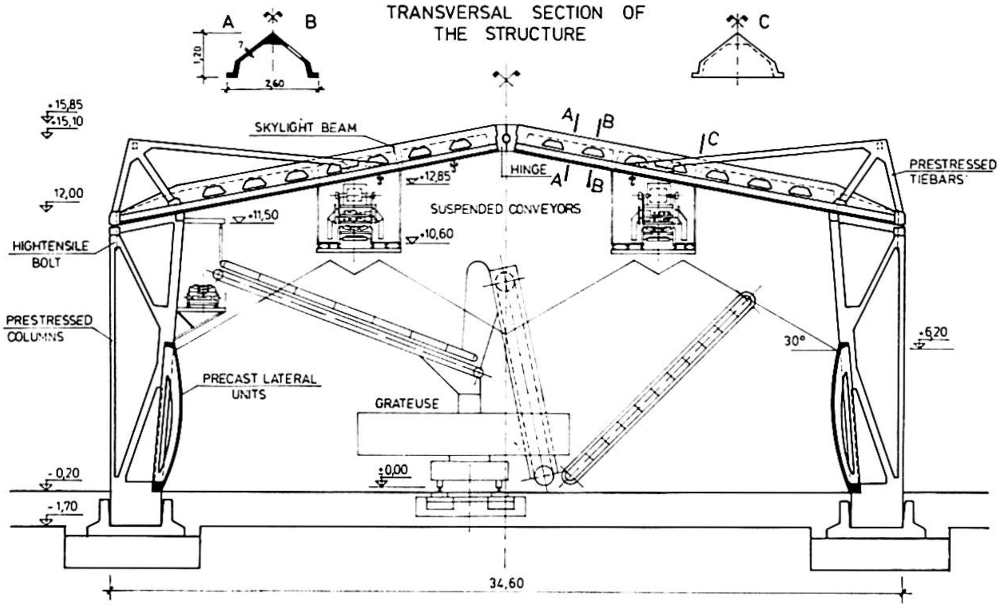
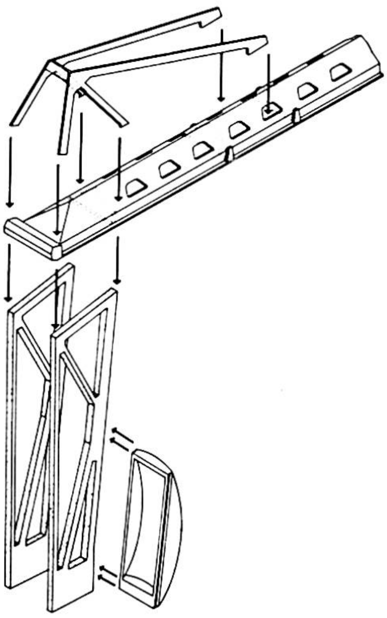
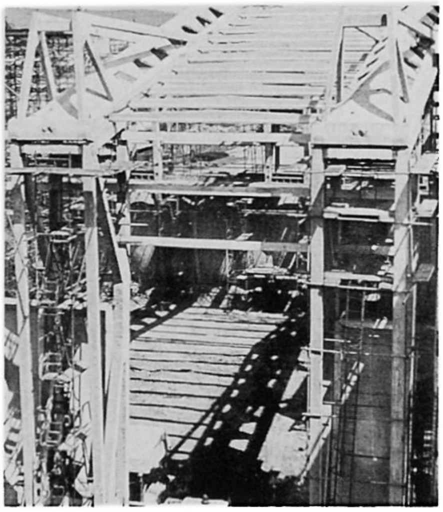


Fig.11. Cross section of the depot structure





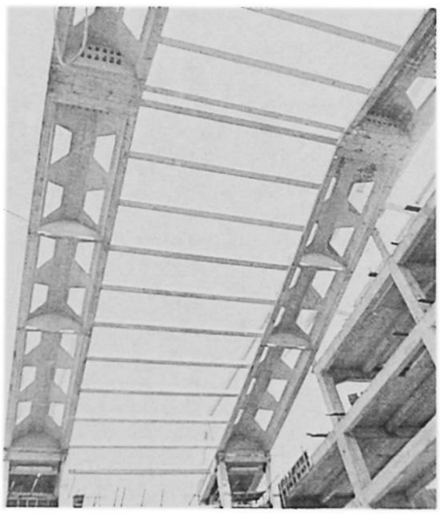


Fig.12. Axonometric view of the precast elements

Fig.13. Ouside view during the erection

Fig.14. Inside view of the precast beam

The spatial development of the main bearing system enabled : (i) a 22 percentage economy in the cladding cement-asbestos sheetings, which cover by pannels only the strips between the beam flanges (ii) a good lateral stability during the erection time assuring meanhile a convenient stiffness in the longitudinal sense to the earthquake action. The main indicators for the entire upper structure are: concrete 0,176 m^3/m^2 and total steel 29,2 kg/m^2.

4. ACKNOWLEDGEMENT

The first structure was advised by Mr.Ing.Mircea Georgescu, the second one has been controlled by Mr.Ing.Ion Găvozdea .

IV

Ninian Central Concrete Gravity Platform

Plate-forme centrale Ninian en béton

Betonschwergewichtsplattform der Ninian Zentrale

J.E. LONG
M.Sc., F.I.C.E., F.I. Struct.E.
PSC Freyssinet Limited
Buckinghamshire, England

SUMMARY
This paper reviews the experiences gained in executing the largest post-tensioning sub-contract ever let in the United Kingdom. Standard systems have been used, but new equipement had to be built to fabricate, handle and thread the very long tendons involved. In particular, the threading and grouting of tall vertical tendons, which is not encountered in routine prestressing contracts, has led to considerable development work. The final successful solution, to overcome the problem of bleeding, is described.

RESUME
Cette contribution examine les expériences acquises lors de l'exécution du contrat le plus important de sous-traitance en post-tension jamais décerné au Royaume-Uni. On a adopté des systèmes standards mais il a fallu construire un matériel nouveau pour fabriquer, manutentionner et enfiler de longs câbles. En particulier, les opérations d'enfilage et d'injection de très grands câbles verticaux qui sortent du cadre habituel des travaux en matière de précontrainte ont amenés à des développements importants. La solution adoptée afin de surmonter avec succès le problème de l'exsudation, fait l'objet d'une description.

ZUSAMMENFASSUNG
Dieser Beitrag berichtet über die Erfahrungen, welche bei der Ausführung des grössten Nachspannungs-Untervertrages, der jemals in Grossbritanien vergeben wurde, gewonnen werden konnten. Herkömmliche Systeme sind benutzt worden, jedoch mussten neue Anlagen konstruiert werden, um die äusserst langen Drähte fabrizieren, handhaben und einfädeln zu können. Insbesondere das Einfädeln und Untergiessen von langen vertikalen Kabeln, was bei gewöhnlichen Vorspannungsarbeiten nicht anzutreffen ist, hat zu erheblichen Entwicklungsarbeiten geführt. Die gewählte, erfolgreiche Lösung zur Überwindung des Abzapfproblems wird beschrieben.

1. INTRODUCTION

Currently, some degree of prestressing is considered essential in offshore gravity structures, partly to save weight and partly in an attempt to minimise cracking, but there are very few special requirements for the prestressing system, unlike nuclear structures for example. Ordinary commercial systems can be used, perhaps slightly modified to permit electrical interconnection for corrosion protection. In terms of modern practice in bridges, the systems employed are universally rather small.

Up to date, companies within the Freyssinet Group have been responsible for providing the prestressing system and equipment for eight deep water gravity platforms, of three basic designs, and have actually carried out sub-contract work on six of these. The biggest by far was the Central Platform for the Ninian Field, built at Loch Kishorn in Scotland, where the prestressing sub-contract executed by PSC Freyssinet Limited was the largest ever let in the U.K. This was the fifth platform built to the Doris design, but the first in the U.K.

A standard 12/15mm strand prestressing system was used throughout, and the sub-contract included the supply of prestressing materials and equipment, tendon fabrication, installation, stressing and grouting, together with bar stressing.

2. PRESTRESSING LAYOUT

The layout of prestressing tendons in the platform structure is essentially as follows :

- Straight horizontal tendons up to 150 metres long in the base
- Curved horizontal tendons up to 140 metres long in the walls, normally anchored at the diaphragms.
- Straight, and some looped, horizontal tendons in the diaphragms.
- Vertical 'U' tendons, up to 75 metres in height, in the walls and diaphragms.

In addition, 8,320 No. 40mm diameter Macalloy bars connect the steel skirt to the main concrete platform structure.

3. DUCTS

Although made from a heavier gauge of steel than in normal bridge work, standard spirally wound prestressing ducting was used horizontally, and this was transported to site by rail, in 6 metre lengths, and connected on site using metal sleeves.

There was extensive use of smooth bore steel tubing, primarily as a construction aid, to facilitate tendon threading around relatively sharp bends, and to ensure a largely self-supporting duct system during slipforming operations, although by virtue of its increased wall thickness, this tube does offer added corrosion protection, and improved resistance to interconnection where ducts cross at 90 degrees and touch.

This tube was pre-bent to the appropriate radius off site and connected with a spigot and socket joint, formed on site. Pre-bending to large smooth curves was carried out on a pipe-bending machine, but occasionaly in-situ adjustments were made by heating the tube locally, when in position.

On tight bends, down to about 1.2 metres radius, the tube was larger in diameter, to facilitate tendon threading, so that special adaptor pieces were produced to join the bends to the normal diameter of tube.

4. TENDON MAKING

Machines do exist capable of pushing strands into ducts directly from the coils. Because of the length of many of the ducts, the speed of construction, the orientation of some of the anchorages, and the need to protect the strand from exposure, such machines were not widely used on this structure, and most tendons were pre-made and then pulled into place. Strand pushing was restricted to a very few ducts, usually in emergencies.

Roughly 3,500 tonnes of strand were factory fabricated into tendons, on a long bed, recoiled and transported to site by rail. There were advantages in this, because an extensive and expensive site installation was avoided, and the tendon making plant is permanent, serving more than one site.

12/15mm strand tendons, up to 150 m long and weighing 2.25 tonnes, were satisfactorily recoiled, with a diameter less than 2.5 metres.

All strand had to receive a coating of soluble oil at the manufacturing plant, for corrosion protection, and this needed renewing during tendon fabrication; consequently, the strands were run through an oil bath, and also the tendons coiled up in store at Loch Kishorn were sprayed at regular intervals with the same oil.

The tendons were all provided with a welded eye on both ends for towing and threading.

5. TENDON THREADING

All ducts were checked for obstructions, usually by blowing through a plunger carrying a light line, which was then used to draw back the main pulling cable. Most horizontal tendons were simply winched in, and quite low forces were involved, perhaps due to the oil on the strand and duct, coupled with the majority of relatively simple tendon profiles. A winch of 1.5 tonne capacity, reacting off the structure itself, was normally adequate; however, looped horizontal tendons did sometimes present problems and required greater forces.

Vertical tendons were threaded from the top downwards, and adequate braking force was needed, which was solved by feeding each tendon from an air powered dispenser, fitted with an automatic brake, to which the tendon was attached by means of the second eye. On 'U' shaped ducts, a winch was used to pull the tendon up the second leg, after it had been carried down the first leg by gravity, or pushed by a dispenser.

6. STRESSING

Stressing presented no new technical problems, but was a formidible exercise in planning and, to avoid constantly moving the equipment, it was necessary to provide 20 jacks and pumps on site. 8 stressing operations were sometimes in progress at the same time, so it was necessary to make a permanent mark on the strand behind the jack, usually by a saw-cut, so that the extension achieved could be checked at any time subsequently, by the Inspectors.

The correlation between calculated load and extension was usually within 5%, presumably due to the workmanship in duct fixing and to the fact that the oil on the strand and duct effectively inhibited any rusting.

With so much activity in a relatively confined area, safety during prestressing operations is obviously a major concern. However, there was no accident attributable to the stressing on this job.

7. GROUTING

Ordinary cement based grout was used throughout, and fairly conventional techniques were used for horizontal tendons, although it was normally impossible to get the grout pump near to the anchorages. Consequently, a central mixing and pumping plant with large diameter delivery lines, up to 300m in length, had to be used, coupled with a retarded grout. In addition, to guard against possible interconnection, several ducts were sometimes grouted simultaneously.

Experience finally showed that it was not necessary to flush out the soluble oil before grouting, it was carried out with the waste grout.

An expanding and water reducing admixture was employed, with a water/cement ratio between 0.34 and 0.40, and it was possible to produce complete filling of the 'U' tubes by pumping from the top, and topping up afterwards, under gravity.

It was known from previous work that bleed water tends to be driven up strands, ahead of the main column of grout, a phenomenon known as the 'wick' effect. It was decided to turn this to advantage, and the strands were therefore left protruding from the top anchorage to encourage bleeding to waste; finally the duct was re-grouted, by gravity at the top anchorage, using a small quantity of retarded grout, which forced the remaining bleed water up the strands.

An anti-bleed additive was investigated, but appeared to offer little advantage with the existing plant, in view of the increased pumping pressures and the greatly increased mixing time involved.

Based upon this experience, the following observations can be made, relating to the grouting of tall vertical ducts.

- Before undertaking the grouting of any job, it is necessary to carry out a trial on site to prove the proposed plant and method, in conjunction with the actual cement. Any reinjection in this trial should be carried out with a coloured grout, to determine the degree of penetration, and then the duct should be cut for inspection. An expanding agent should be employed.

- The maximum grouting pressure will sometimes need to be a good deal higher than in horizontal work, and this may require special treatment at the anchorage where grouting is carried out. Much lower pressures are involved in grouting 'U' tubes, thus minimising bleeding.

- Strands should be allowed to protrude through the grout seal on the top anchorage to facilitate the wick effect. The bleeding from these strands will probably be in excess of that measured in a conventional test in a cylinder, due to the pressure filtration effect up the strands during grouting.

- Previous researchers have cast doubt on the value of the practice of holding pressure on the grout column for a period after injection. Here this has not been done, with no discernible detrimental effects, so it appears to be a time-consuming practice that can be abandoned.

- Reinjection may be carried out from the top, within an hour after the completion of the initial grouting, most simply by installing a small header tank containing 15 - 20 litres of fresh retarded grout about 1 metre above the top anchorage to provide a continuous reservoir. Bleeding from strands will then occur anew.

8. RECORDS AND QUALITY CONTROL

Very detailed records had to be kept, of all aspects of the prestressing work, so that the complete case history of any tendon in any duct was known. It had to be possible to trace a strand from manufacture, through tendon making, to tendon threading, stressing and grouting. All prestressing materials were subjected to inspection by the Certifying Authority.

Routine quality control tests were carried out on the grout, for fluidity and bleeding. Fluidity was measured with a flow cone, which is a simple and acceptable site test, although unfortunately not yet standardised completely so that cones of different dimensions are encountered. The time taken to fill a one litre container, from a full cone, is taken as the flow time, and, times in the approximate range of 12 seconds - 30 seconds were normally satisfactory, grouts with higher flow times tended to block in the pumping mains and those with lower flow times exhibited excessive bleeding. This test is probably best used as a basis of comparison of fluidity, once a satisfactory mix has been established.

Both the limits of bleeding and the method of test were generally those recommended by the F.I.P. viz : the bleeding at 20 degrees C, of a sample of grout 100mm diameter and 100m high, must not exceed 2% at 3 hours after mixing, with an absolute maximum value of 4%, and complete re-absorbtion of all bleed water after 24 hours. With heavily retarded grout, this reabsorbtion requirement was difficult to achieve, and was relaxed to 48 hours, to allow for complete setting of the grout.

9. DOCK GATES

The two concrete gates to the dry dock, form part of the development of the site facilities by Howard-Doris Limited, and, in any other location, would be regarded as a considerable job in themselves, although they were dwarfed by the adjacent platform. Each gate is roughly 83 metres long x 14 metres wide x 15 metres high, and is composed of 24 cells formed by two internal longitudinal walls and seven internal cross walls.

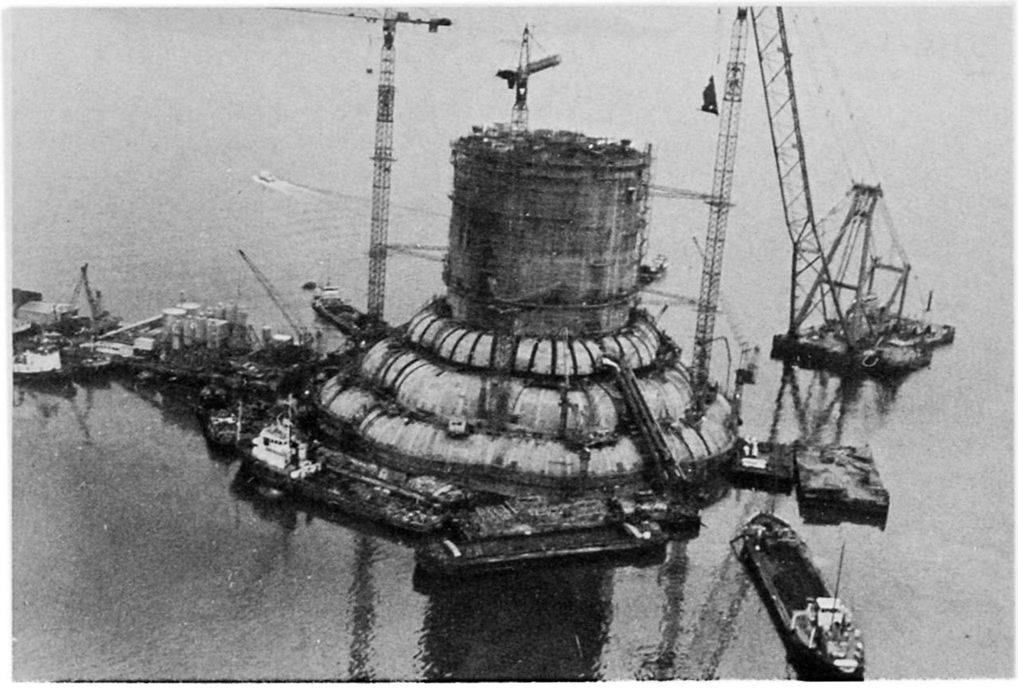
1647 No. 25mm diameter Macalloy bars were used for the vertical prestressing, with 530 No. 32mm diameter Macalloy bars transversely. For the longitudinal prestressing, the same 12/15mm strand system was used as on the main platform, with the tendons on the sea side overlapping at the central wall, and stressed internally, to avoid the use of stressing anchorages on the ends of the gates.

ACKNOWLEDGEMENTS

Client : Chevron Petroleum (U.K.) Limited

Main Contractor : Howard-Doris Limited

Prestressing Sub-Contractor : PSC Freyssinet Limited



IV

Caissons with Prestressed Rock Anchors as Soil Retaining Structures

Structures en caissons avec ancrages en rocher précontraints pour contenir la poussée des terres

Vorgespannte, felsverankerte Caissons als Erd-Sicherungs-Bauwerk

T.C. LIAUW
Senior Lecturer
University of Hong Kong
Hong Kong

SUMMARY
To overcome the difficulties of construction development in the hilly terrain of Hong Kong, a new form of soil retaining structure has recently been evolved. The retaining structure consists of caissons, either contiguous of spaced with laggings, tied back by temporary or permanent prestressed cables anchored into the solid rock stratum. The design principles and construction procedures are described in connection with an actual site for a tall building requiring such a solution.

RESUME
Afin de résoudre les problèmes qui se posent dans la réalisation de constructions sur le terrain accidenté de Hong Kong, une nouvelle forme de structure pour retenir les terres a été développée récemment. La structure de fixation est composée de caissons qui sont soit contigus, soit écartés avec des garnitures et ancrés par des câbles précontraints de manière temporaire ou permanente dans le rocher solide. Les solutions de procédés de construction donnés sont illustrés dans le cas d'un bâtiment élevé nécessitant une telle solution.

ZUSAMMENFASSUNG
Um die Schwierigkeiten bei Bauvorhaben in Hong Kongs hügeligem Gelände zu überwinden, wurde vor kurzem eine neue Methode zur Sicherung des Erdreichs entwickelt. Das Sicherungsbauwerk besteht aus Caissons, die entweder dicht aneinander gereiht sind, oder deren Zwischenräume ausgefacht sind. Die Befestigung erfolgt durch zeitweilig oder dauernd vorgespannte, in festen Felsschichten verankerte Kabel. Es werden die Entwurfsprinzipien sowie der Konstruktionsablauf anhand einer Baustelle für ein hohes Gebäude, welche eine solche Lösung verlangte, dargestellt.

1. INTRODUCTION

Hong Kong with a total land area of only 1,052 square kilometres is one of the most densely populated place in the world. The total estimated population at the end of 1978 was 4.7 million excluding refugees and illegal immigrants, making an average density of 4,500 per square kilometre with the high density of 25,000 per square kilometre in the urban areas and the low density of 500 per square kilometre in the suburb.

As the flat land in the urban areas is very scarce and very expensive, further development in recent years has spread to more and more difficult hilly terrain. Many site formations on steep slopes of residual soils pose precarious situation and some of them had caused minor and mojor landslides despite the usual measures being undertaken. Therefore, development in the hilly terrain has been hampered and the Government has imposed moratorium restricting new development in certain area. The pressure of land cost and population expansion creates the need to utilize even the 'undevelopable' land and to open up new towns in the suburb. This paper is concerned with the former aspect in which a newly evolved construction technique is used to make development possible in very difficult terrain.

2. SOIL RETAINING CAISSONS

2.1 Design Principles

Caisson has been used primarily for the transmission of vertical load to the deep strata. Unlike the ICOS wall which requires heavy machinery for excavation, the excavation for large-diameter caisson is carried out by hand-digging technique using simple tools to overcome varying sub-stratum from soft to hard, particularly when boulders are encountered. The excavation for caissons can, therefore, be carried out not only on flat ground for deep foundation transferring vertical load, but also on hilly terrain. This opens to the possibility of caissons being used as retaining members resisting horizontal earth pressure in bending.

However, cantilevered caissons can not resist much soil pressure especially in hilly terrain where there is large surcharge from the slope. In order to make the caissons capable of resisting considerable soil pressure as a retaining wall, they are tied back along the height by prestressed cables which are anchored into solid rock (Fig. 1). These rock anchors may be a temporary measure if permanent horizontal props (such as floor slabs) can be established later on, or they can be a permanent measure if permanent horizontal props can not be established such as in the case of a slope retaining caissons considered in the example shown later. With these rock anchors, the anchor heads on caisson become the supporting points and the caisson becomes a member resisting horizontal pressure in bending and in compression if there is vertical load.

There are two important aspects in the design of the caissons:
The first aspect is the earth pressure which directly affects the bending moment in the caisson. It is affected by the amount of the prestressing forces due to the soil-structure interaction. The problem is highly indeterminate which may require finite-element analysis together with on-site adjustments of the prestressing forces as relaxation might occur in the process of stressing sequence.
The second aspect is the direction and inclination of the prestressing cables which normally have to be steeply inclined downward in order to anchor them into the rock. This will produce eccentric vertical components of the prestressing forces at the anchorage heads with the resulting increases in bending moment.

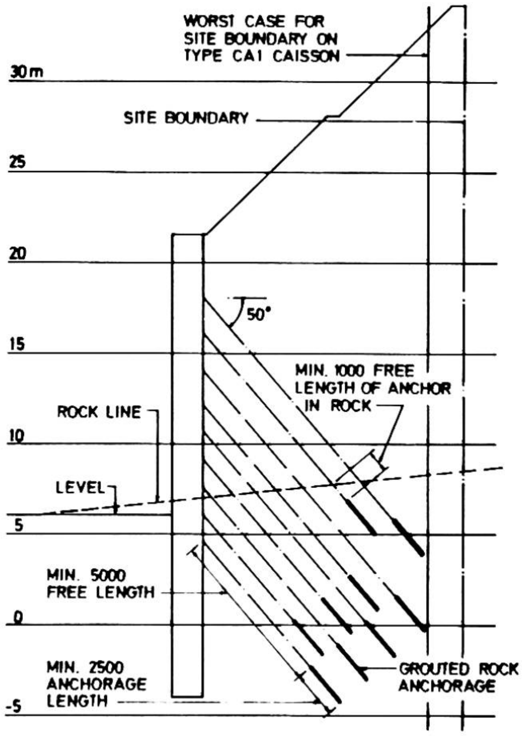


Fig. 1 Cross-Section Through Retaining Caisson

2.2 Construction Sequence

The construction sequence described in the following is related to an actual site. Before the excavation for the caissons, the first stage site formation (Fig. 2) started with slope cutting and temporary drainage in order to facilitate the construction of the caissons, the delivery of materials and transportation of spoils. The excavation for each caisson was carried out in stages of one metre deep approximately, followed by the casting of concrete ring. While excavation deepened, these concrete rings formed the shield of the shaft. Boulders were broken up whenever they were encountered by the use of neumatic drill. Having formed the shaft down to bed rock, the caisson reinforcement was lowered into the shaft and lagging starter bars were fixed through drilled holes in the shaft. Steel pipes, bearing plates, anchor block spiral reinforcement and box-out blocks were positioned from the inside of the shaft. The caisson was then concreted using tremie tube.

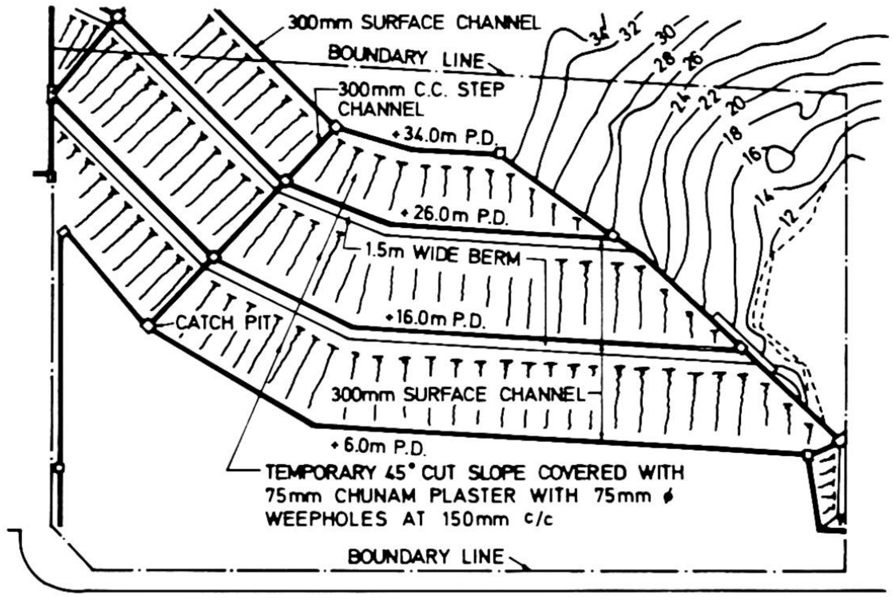


Fig. 2 First-Stage Site Formation

The second stage site formation, (Fig. 3), followed the completion of caissons. The soil was excavated on the outside of caissons which were exposed to the first level of anchorage positions and the anchorage pockets were made good. Using the embedded steel pipes as alignment guides, (Fig. 4), the anchor holes were drilled into the rock. The tendons, with their free length coated with grease and wrapped with polythene tape, were placed in positions and grouted for the required length in rock for bond (Fig. 5). Water test was carried out to ensure each grouting had been properly carried out and no leakage occured. The tendons were then stressed to the predetermined forces and locked at the anchorage heads. The process from excavation to stressing of tendons was repeated until the full depth was reached.

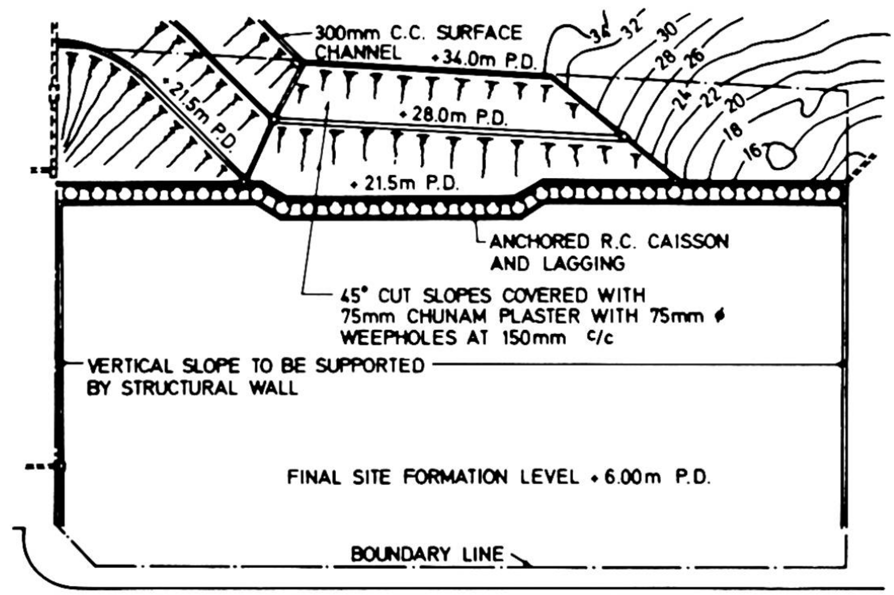


Fig. 3 Second-Stage Site Formation

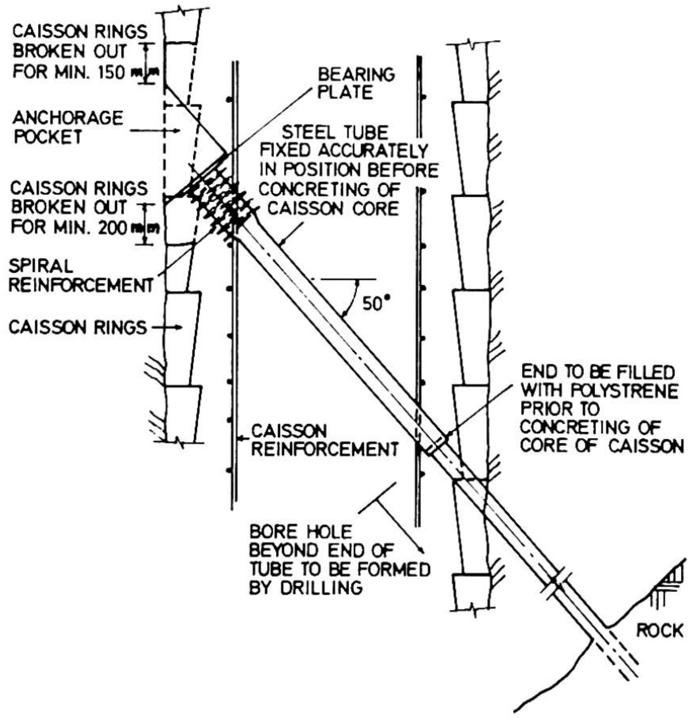


Fig. 4 Upper Portion of Rock Anchor

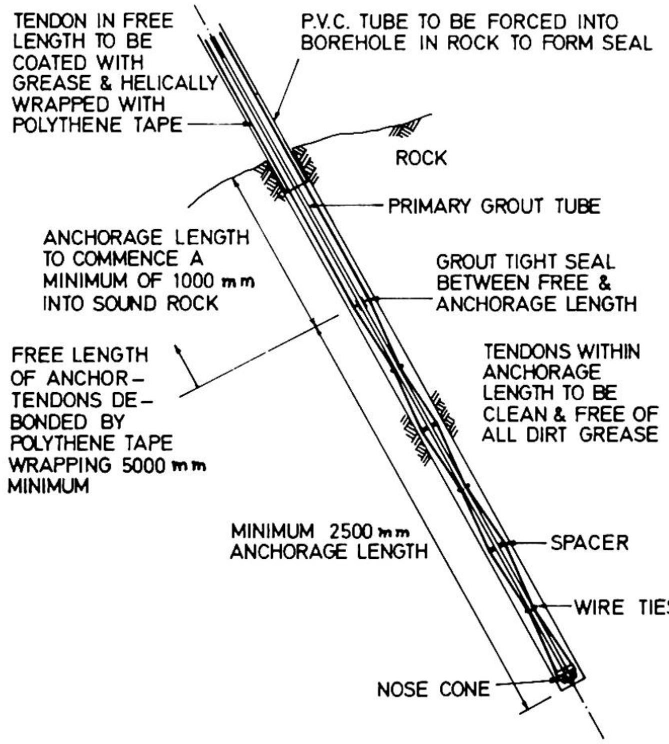


Fig. 5 Lower Portion of Rock Anchor

3. FIELD INSTRUMENTATION

Because of the importance of the retaining structure to the project and the lack of information regarding soil-structure interaction of this nature, field instruments were installed to collect and monitor field data in order to check the design parameters and assumptions, and to observe any sign of abnormality so that steps can be taken early to rectify dangerous situation if it arises.

The field instruments installed on site (Fig. 5 & 6) are briefly described as follows:

(a) Piezometers: installed around the site to monitor pore water pressure in the soil for the evaluation of slope stability.
(b) Inclinometers: plastic casings were installed in the retaining structure, from the top of which the sensor can be lowered into the casings for the measurement of slope deformation of the structure.
(c) Earth pressure cells: installed at different levels and locations behind the retaining structure to monitor the actual earth pressure on the structure.
(d) Load cells (annular ring type): installed between the anchor heads & bearing plates of some anchorages to monitor the variation of cable forces.

The actual field data collected through these instruments provide the basis to correlate the loading on the structure with the effects, and enable the checking of the design and safety of the structure.

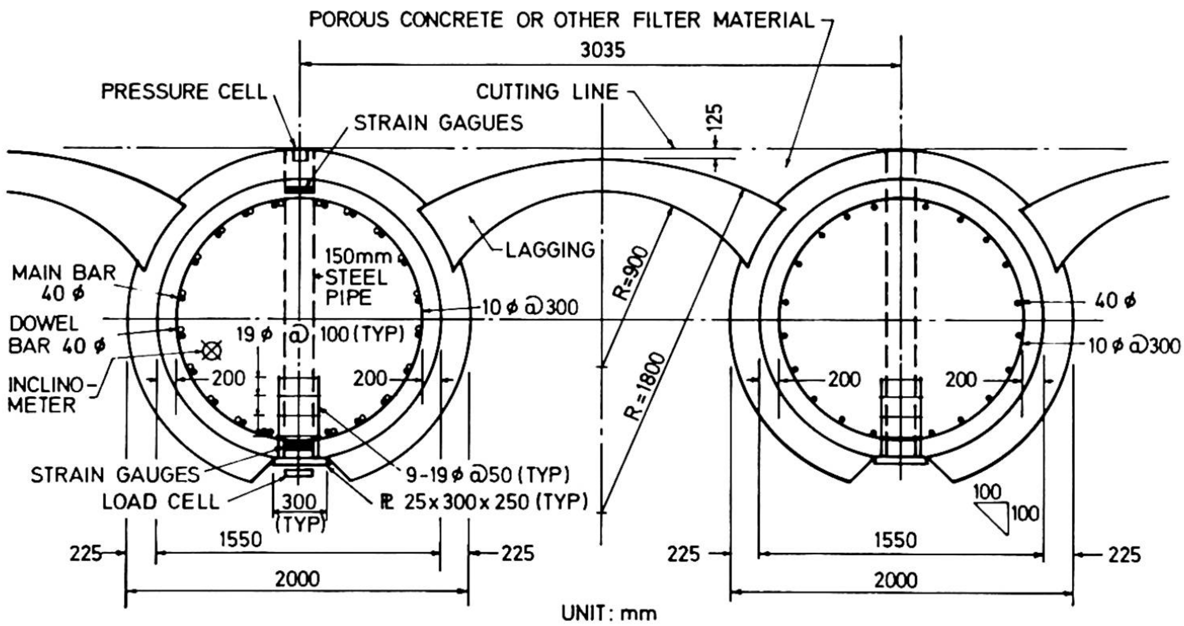


Fig. 6 Locations of Field Instruments

4. CONCLUDING REMARK

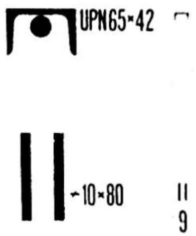
A type of soil retaining structure has been evolved in the form of caissons tied back by prestressed cables which are anchored into solid rock. The use of such retaining caissons enables the formation of difficult site in the hilly terrain where heavy machinery can not be employed. Field instrumentation produces useful data which in turn provide the confidence in the design and safety of the structure.

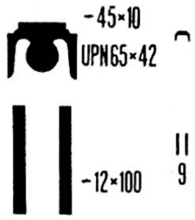
UNIVERSITA' degli
STUDI di BARI
FACOLTA' di INGEGNERIA
ISTITUTO
di
SCIENZA delle COSTRUZIONI

viale Japigia, 182 Bari (Italy)

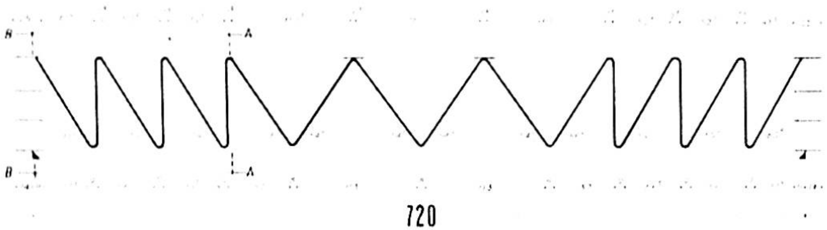
TRIALS ON BEAMS IN METAL TRESTLE BURIED IN CONCRETE

G. DONATONE – G. FRADDOSIO – A. SOLLAZZO

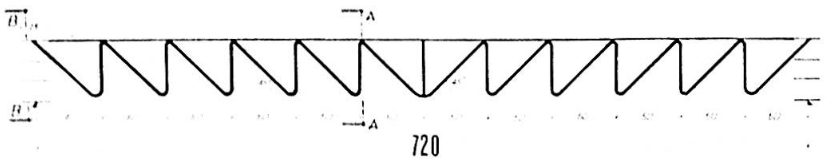




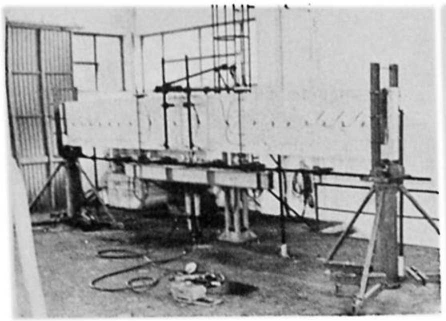
TRESTLE OF THE BEAMS 1–2

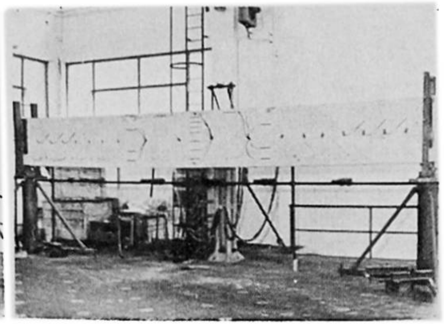


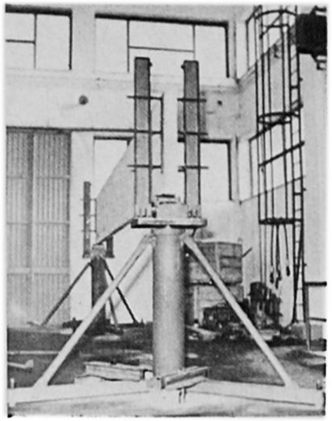
TRESTLE OF THE BEAMS 3–4



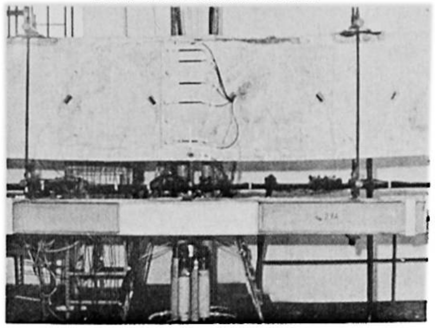
LOAD DEVICES AND BEAM AFTER CRACKING



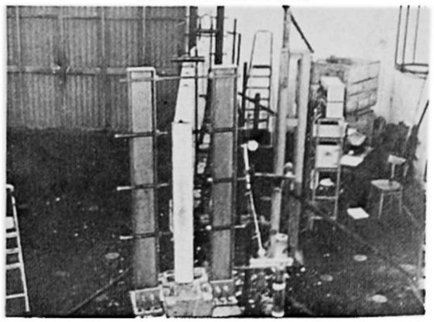


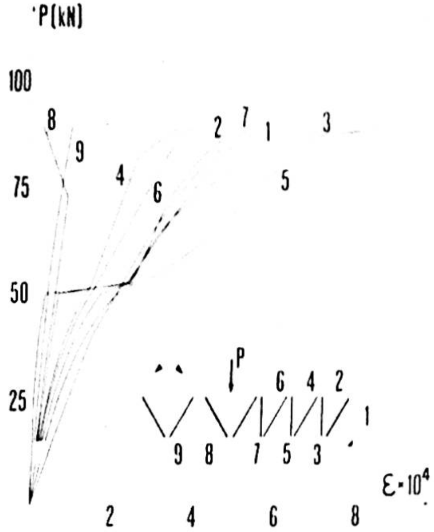


BEAM WITH BEARING DEVICES

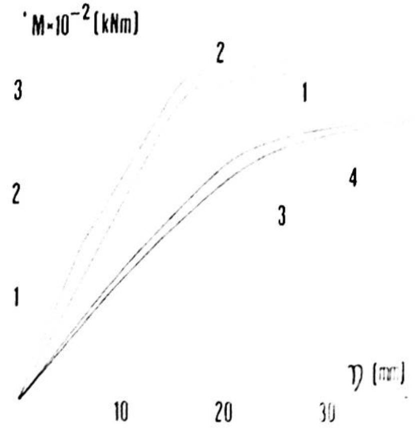


BEAMS AFTER FAILURE





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.

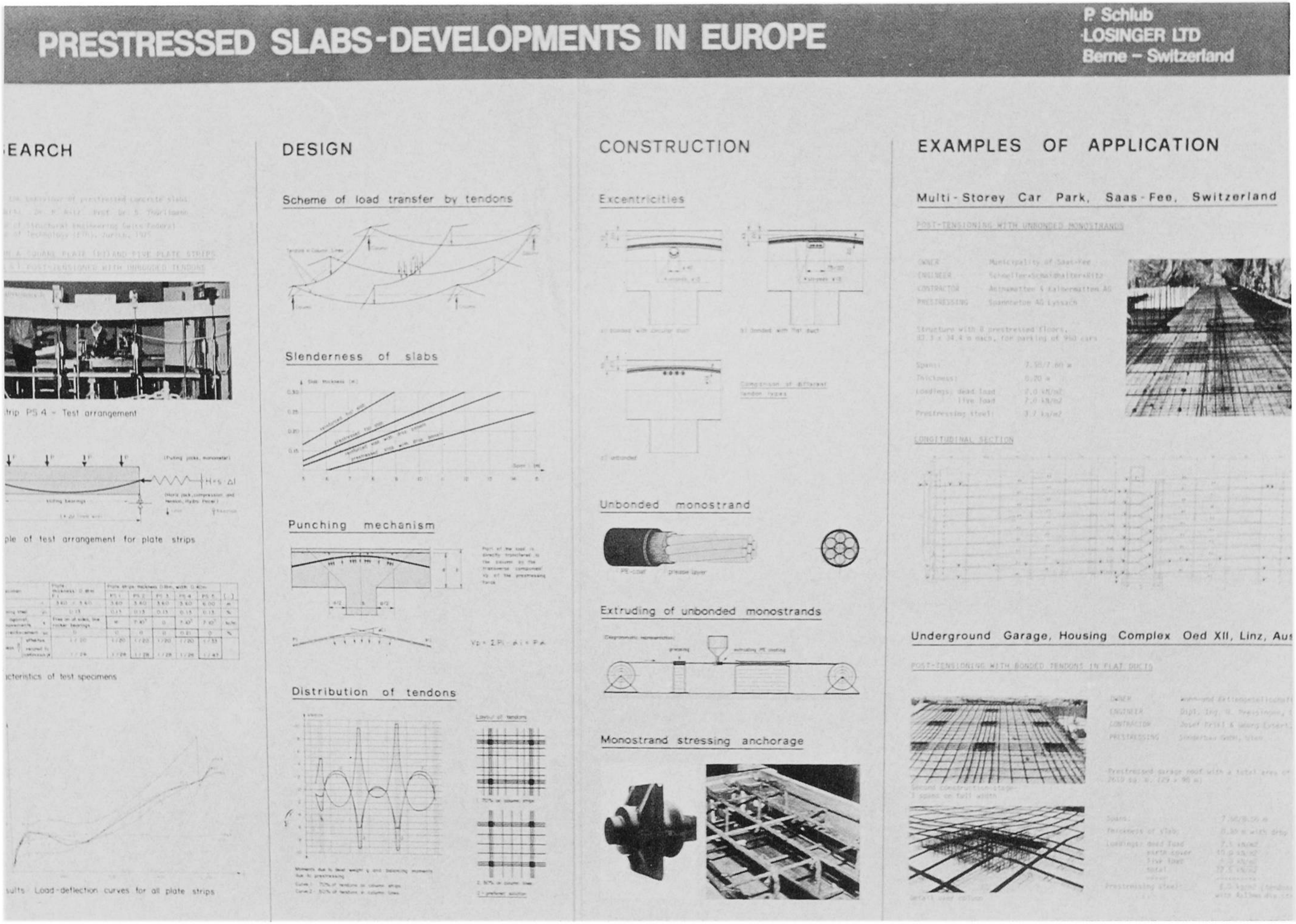
Special "diapason" bearings have been designed to prevent only the beam rotation around its longitudinal axis and loads have been applied by means of previously calibrated hidraulic jacks.

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. Schlub
LOSINGER LTD
Berne – Switzerland
EARCH
DESIGN
CONSTRUCTION
EXAMPLES OF APPLICATION
Scheme of load transfer by tendons
Slenderness of slabs
Punching mechanism
Distribution of tendons
Excentricities
Unbonded monostrand
Extruding of unbonded monostrands
Monostrand stressing anchorage
Multi-Storey Car Park, Saas-Fee, Switzerland
Underground Garage, Housing Complex Oed XII, Linz, Aus
trip PS 4 – Test arrangement
ple of test arrangement for plate strips
cteristics of test specimens
sults Load-deflection curves for all plate strips

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.

<u>Unbonded</u>

- 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

<u>Bonded</u>

- 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

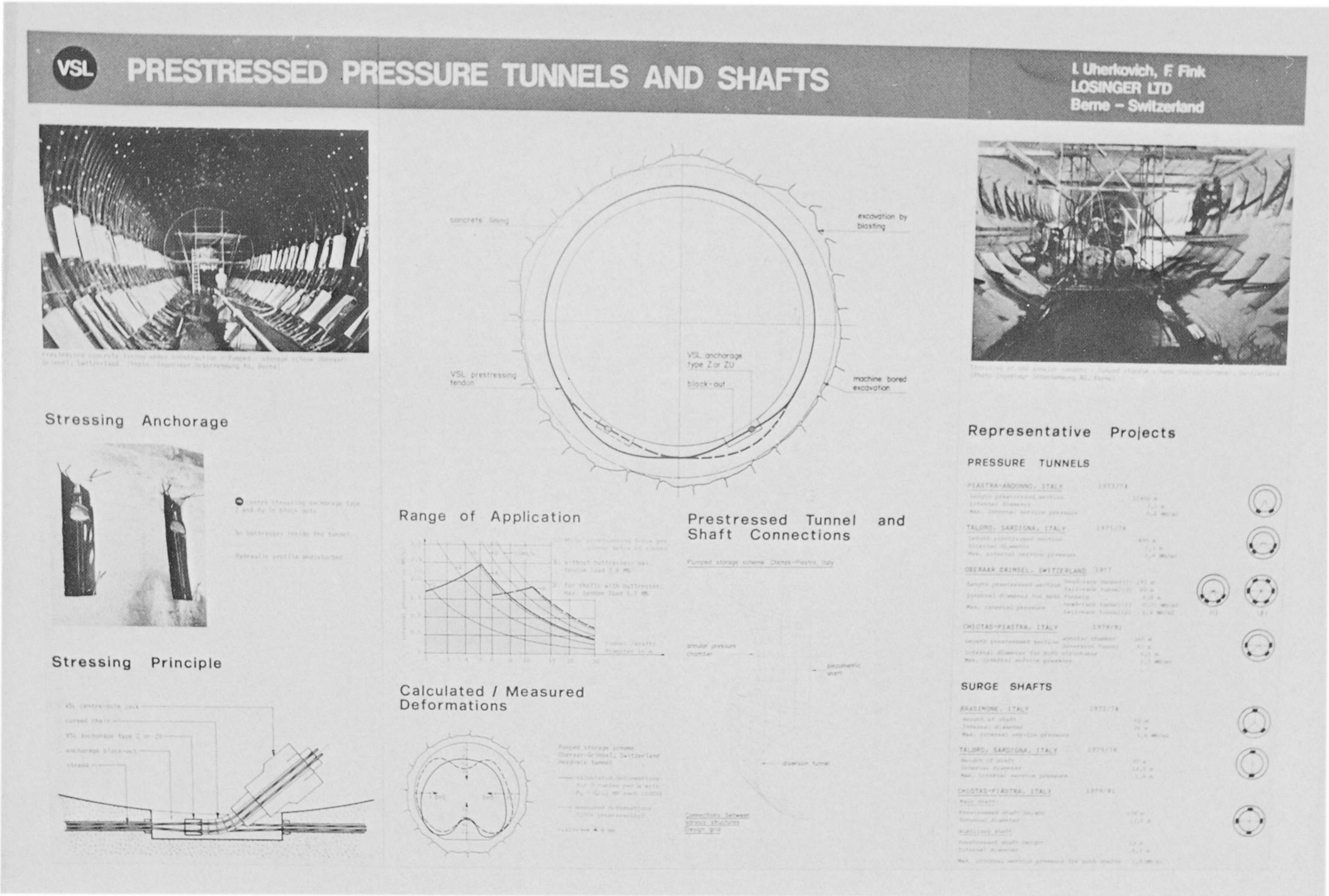
References:

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"


VSL
PRESTRESSED PRESSURE TUNNELS AND SHAFTS
I. Uherkovich, F. Fink
LOSINGER LTD
Berne – Switzerland
concrete lining
excavation by blasting
VSL anchorage type Z or ZU
block-out
VSL prestressing tendon
machine bored excavation
Stressing Anchorage
Stressing Principle
Range of Application
Calculated / Measured Deformations
Prestressed Tunnel and Shaft Connections
Representative Projects
PRESSURE TUNNELS
SURGE SHAFTS

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 contract 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.

ASTM

A unique system of High-Rise Residential Buildings by Large Steel Structural Framework

Toshiharu Hisatoku*
Ryoji Tamura**
Yuzo Kato***

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.

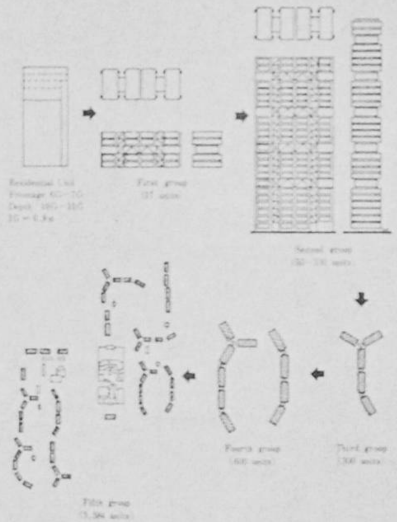


Fig. 1 System of High-Rise Residential Buildings

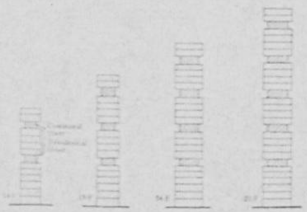


Fig. 2 Composition of Stories

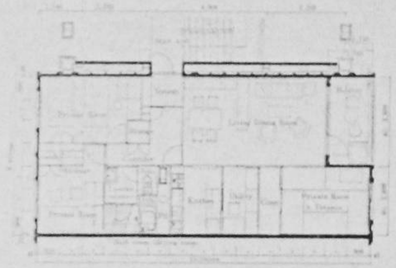


Fig. 3 Plan of A Residential Unit

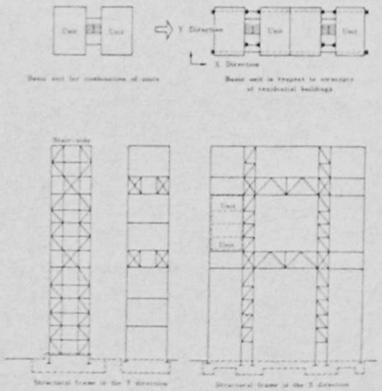


Fig. 4 Structural Framework

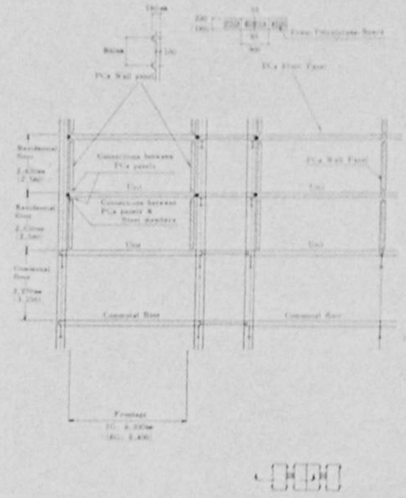


Fig. 5 Structure of Residential Units

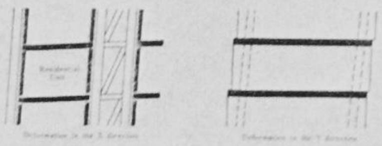


Fig. 6 Deformation of Residential Units

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

T. HISATOKU, Dr. Eng.
Chief Structural Engineer,
Takenaka Komuten Co., Ltd.
Osaka, Japan

R. TAMURA
Chief Structural Engineer,
Nippon Steel Corporation.
Tokyo, Japan

Y. KATO
Chief Structural Engineer,
Takenaka Komuten Co., Ltd.
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

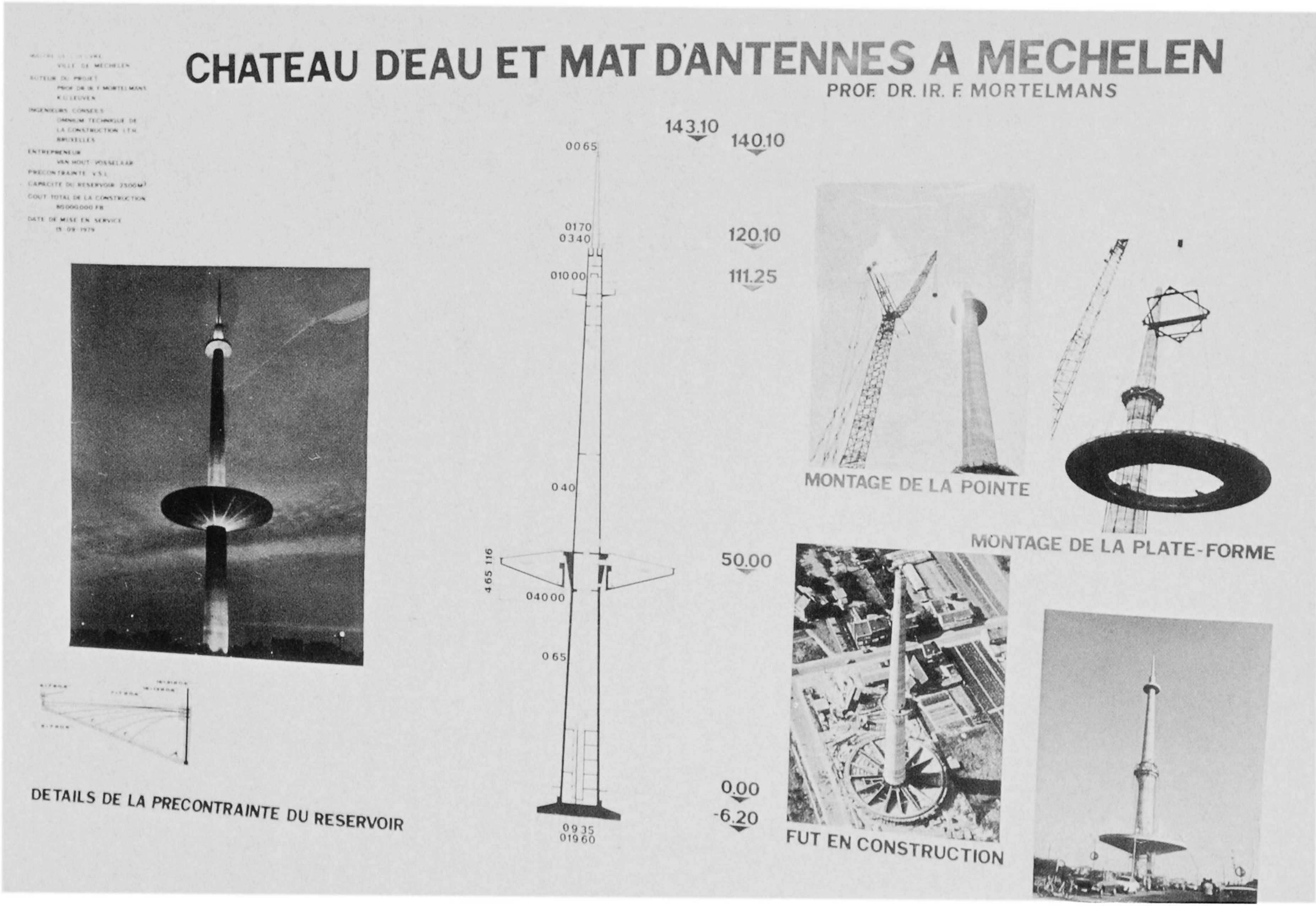
The structural frame of the residential buildings are shown in Fig. 4. The basic unit concerning the structure is four residential units per floor as shown in the figure. In the X direction (see Fig. 4) in order to create the free space for residential units, structural frame consist of two large rigid 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 flamework 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
VILLE DE MECHELEN
AUTEUR DU PROJET
PROF DR IR F MORTELMANS
K.U.LEUVEN
INGENIEURS CONSEILS
OMNIUM TECHNIQUE DE
LA CONSTRUCTION
BRUXELLES
ENTREPRENEUR
VAN HOUT VOSSELAAR
PRECONTRAINTE V.S.L.
CAPACITE DU RESERVOIR 2500M³
COUT TOTAL DE LA CONSTRUCTION
80.000.000 FB
DATE DE MISE EN SERVICE
15-09-1979
143.10
140.10
120.10
111.25
50.00
0.00
-6.20
MONTAGE DE LA POINTE
MONTAGE DE LA PLATE-FORME
FUT EN CONSTRUCTION
DETAILS DE LA PRECONTRAINTE DU RESERVOIR

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 m^3 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.
Apparamment 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
Systhème de précontrainte et de levage : V.S.L.
Pieux des fondations : Soc. Pieux Francki
Entrepreneur : Soc. Van Hout à Vosselaar (Belgique)
Coût des travaux : 80.000.000,- FB
Délai d'exécution des travaux : 200 jours ouvrables

6D-Bauverfahren · Doubrava KG · Attnang · Austria

Elementierter Stahl-Hochbau mit hohem industriellen Vorfertigungsgrad

In- und Auslandspatente

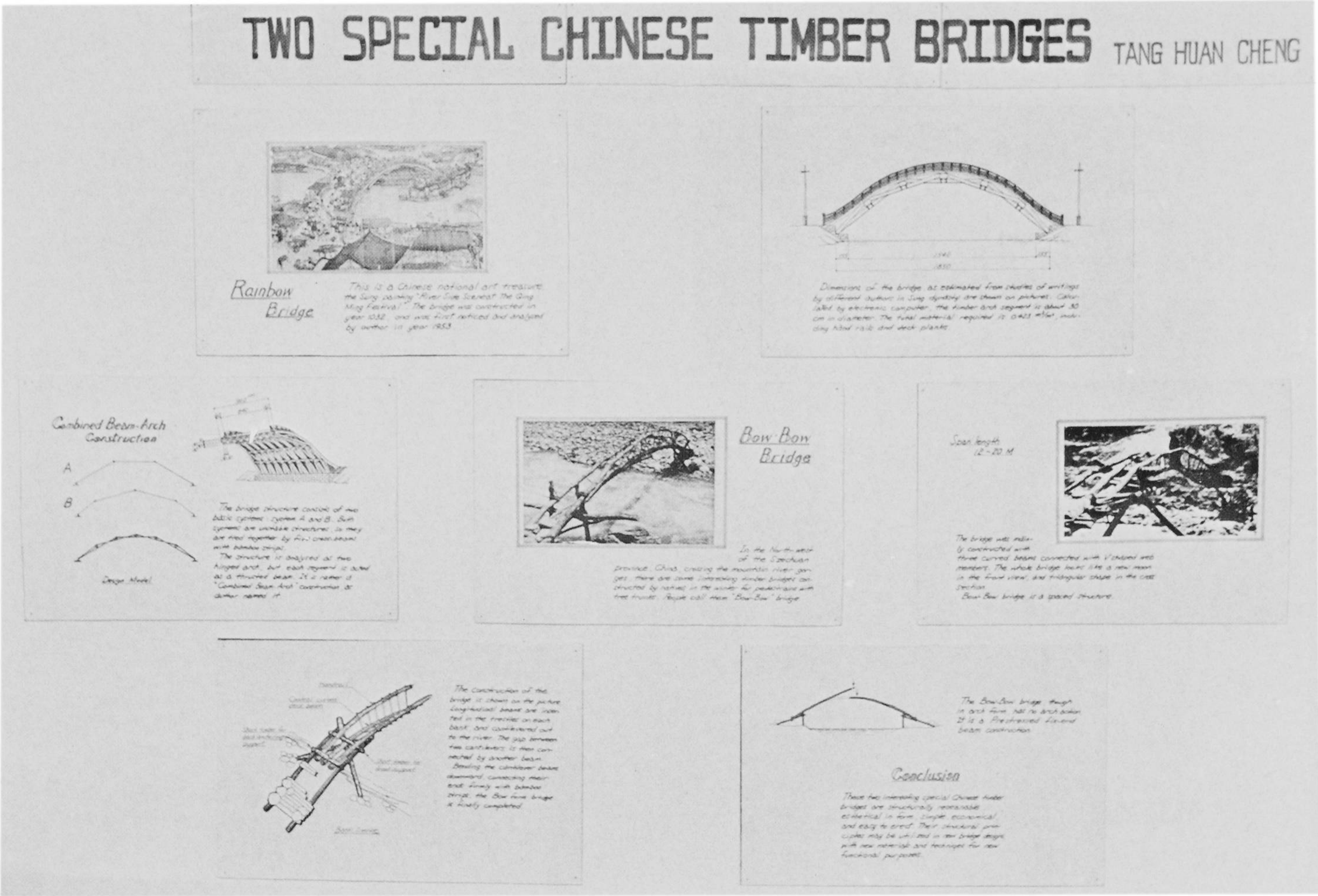
BAUEN OHNE GERÜST

Die Entwicklung des 6D-Bauverfahrens stand unter dem Protektorat der Österr. 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 Sung painting "River Side Scene at the Ging Ming Festival". The bridge was constructed in year 1032, and was first noticed and analysed by author in year 1953.
Combined Beam-Arch Construction
A
B
Design Model
Bow-Bow Bridge
In the North-west of the Szechuan province, China, crossing the mountain river gorges, there are some interesting timber bridges constructed by natives in the winter for pedestrians with tree trunks. People call them "Bow-Bow" bridge.
Span length 12-20 M
Conclusion

prof. dr kruno
TONKOVIĆ

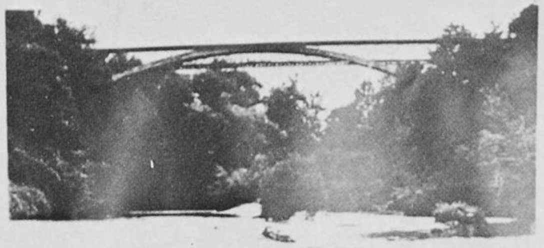
zagreb - jugoslavija
građevinski institut

4 STRUCTURES EN BOIS ◦ WOOD ◦ HOLZ ◦ ◦

JUGOSLAVIJA

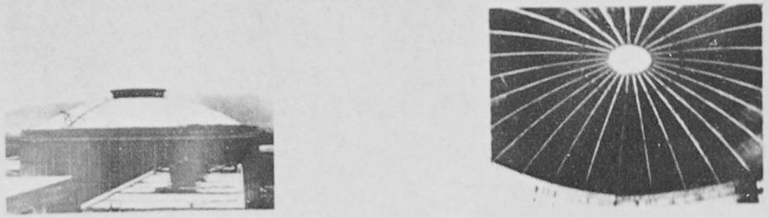
IVBH
IABSE
AIPC

1980

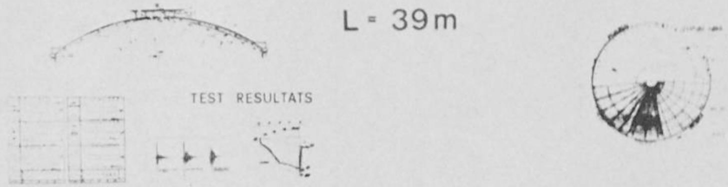


85m

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

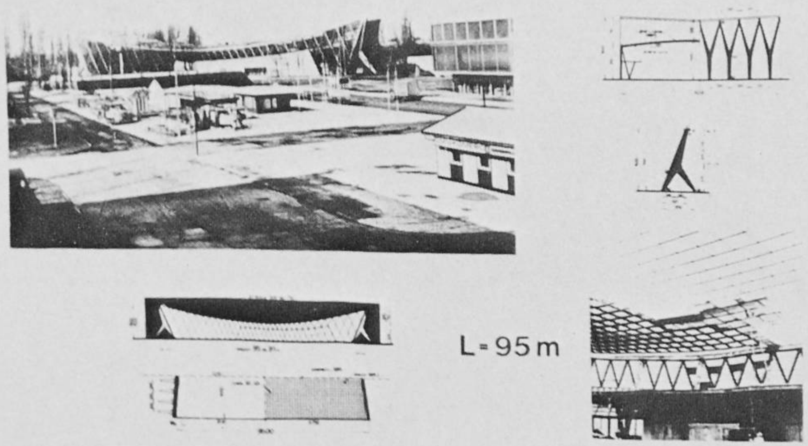


L = 39m

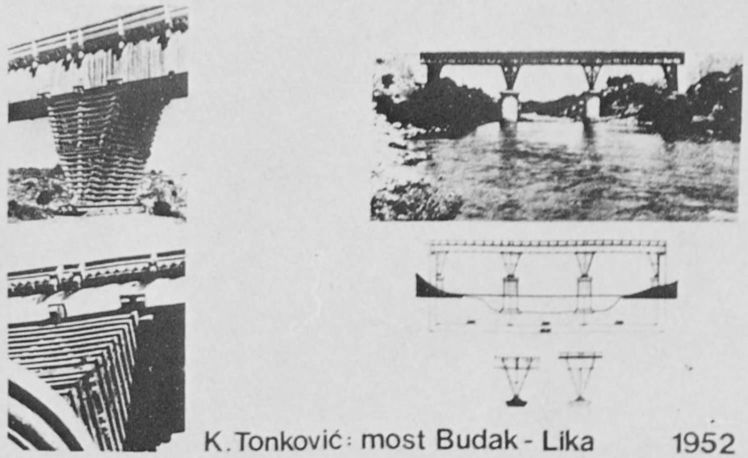


K. Tonković: Kupola brodarski institut - Zagreb - 1954

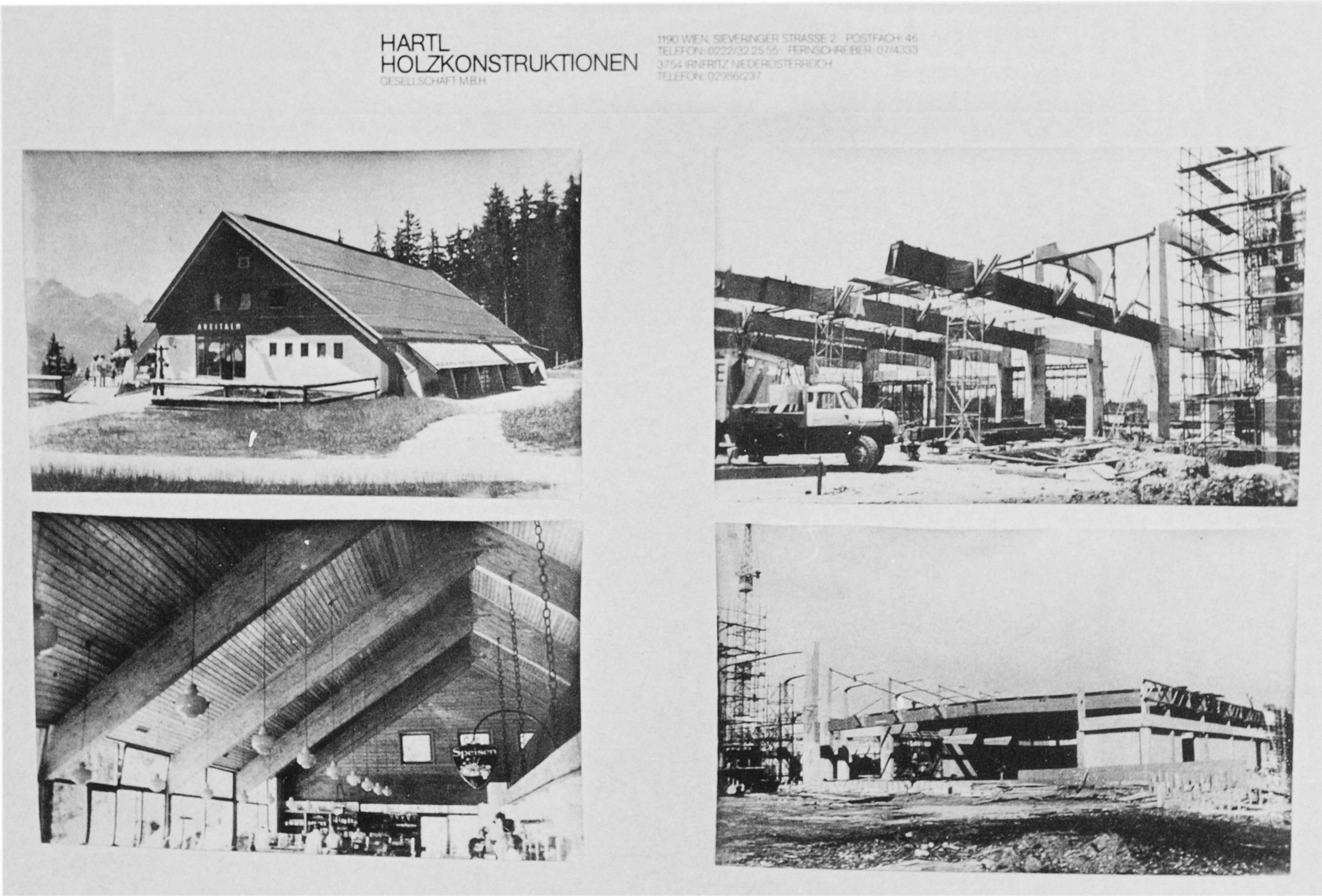
K. Tonković: hala velesajam - Zagreb 1955



L = 95m



K. Tonković: most Budak - Lika 1952


HARTL
HOLZKONSTRUKTIONEN
GESELLSCHAFT M.B.H.
1190 WIEN, SIEVERINGER STRASSE 2 · POSTFACH: 46
TELEFON: 0222/32 25 55 · FERNSCHREIBER: 07/4333
3754 IRNFRITZ NIEDERÖSTERREICH
TELEFON: 02986/237
Speisen