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POSTER SESSION 2

Innovative Building Structures

Structures innovatrices de bâtiments

Neuartige Hochbauten

Coordinators: A. Sarja, Finland
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Mehrfachnutzung des städtischen Bodens durch Gebäude

Multifunctional Use of Urban Space through Buildings

Meilleure utilisation de l'espace urbain par des bâtiments

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In der Zukunft ist mit einer starken Zunahme der Weltbevölkerung zu rechnen. Viele Menschen werden dann in städtischen Ballungsgebieten ihre Existenzmöglichkeit suchen. Zur Vermeidung der Zersiedlung der Landschaften ergibt sich damit die Notwendigkeit, den vorhandenen Boden rational und multifunktional zu verwerten.

In gemäßigten Klimazonen sind vor allem terrassierte Bauten in humaner und ökologischer Hinsicht besonders wertvoll [1]. Mit einer Bepflanzung sind die Terrassen zusammen mit Gründächern Ersatz für die ursprünglichen Vegetationsflächen der Landschaft. Den Bewohnern bieten sie als individuelle oder gemeinschaftliche Freiräume eine höhere Lebensqualität. Es ist möglich, sie zu Wintergärten oder Gewächshäusern auszubauen. Großflächige Flachdächer lassen sich zu Erholungsstätten herrichten. Auch sind auf ihnen solare Wärmegewinnungsanlagen zu installieren, die freistehend der Sonne, dem Wind und dem Regen ausgesetzt sind und womit ein ökonomischer Effekt zu erzielen ist.

In diesen terrassierten Bauten sind großräumige, familiengerechte Wohnungen zu erstellen, wenn sie in der Schottwandbauart errichtet werden, wobei Stahlbetonhohlplatten als Geschoßdecken auf den im Abstand von 7 bis 14m befindlichen Stahlbetonschottwänden aufliegen. Durch Auflösung der Schottwände in Skelett- oder Rahmenkonstruktionen sind in den Geschossen hintereinanderliegende Räume zu Großräumen auszubilden. In den Geschoßdecken sind Installationsleitungen und Tragbaken deckengleich zu verlegen, sodaß eine flexible Gestaltung der Räume ermöglicht wird.

Die oberen, terrassierten Geschosse stehen für die Wohnanlage zur Verfügung. In den unteren Geschossen mit den größeren Räumen und Terrassen sind Gemeinschaftseinrichtungen wie Kindergärten, Krankenpflegestationen, Clubräume für Senioren, Räume für kulturelle oder sportliche Aktivitäten, aber auch für Cafés und Restaurants einzubrin-

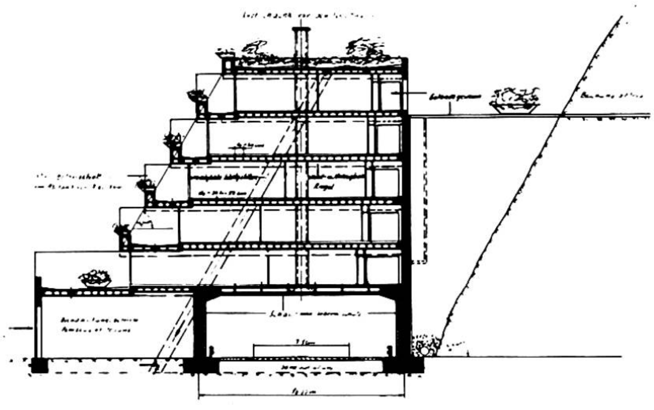

Fig.1 Straßendurchführung durch ein Terrassenhaus am Hang.

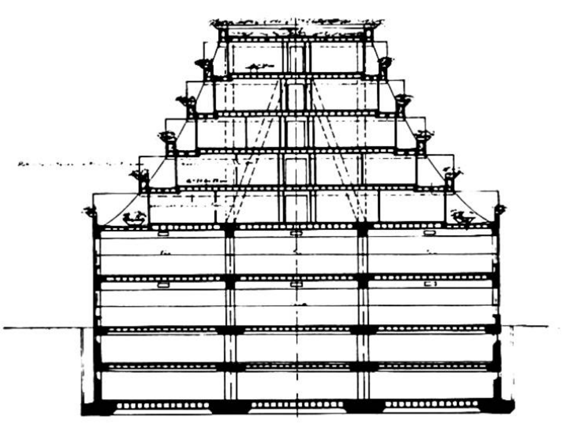

Fig.2 Steilwandiges Gebäude mit oberem Terrassenaufbau.

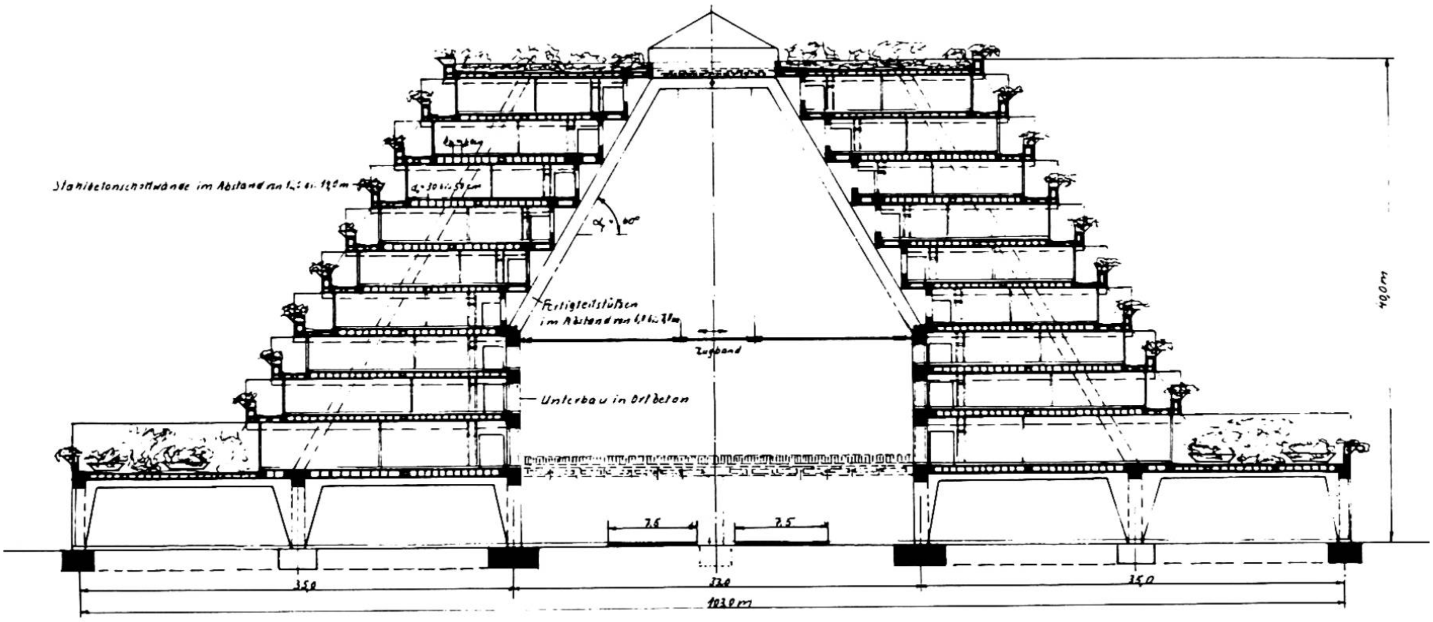

Fig.3 Vielgeschossiges Terrassenbauwerk auf einer Stützkonstruktion.

gen. Die großräumigen Erdgeschosse eignen sich zur Unterbringung von Dienstleistungsbetrieben, Geschäften, Werkstätten und Fahrzeugen. Mit den weitgespannten Decken ist es auch möglich, durch das Keller- oder Erdgeschoß dieser ein- oder beidseitig terrassierten Bauten Verkehrswege oder -straßen zu führen.

An Hand der aufgeführten Querschnittsskizzen soll dargelegt werden, wie mit solchen Gebäuden eine Mehrfachnutzung der städtischen Grundfläche stattfinden kann:
Werden einseitig terrassierte Bauwerke vor einem Berghang (Fig.1) errichtet, wirken sie zunächst wie eine Stützmauer zur Böschungssicherung an dem Hang. Mit ihrer breiten Basis sind sie in der Lage, auch größere Erddruckkräfte bei einer Hinterfüllung am Hang abzufangen, sodaß mit ihnen oben hangseitig ein größerer Geländegewinn zu erzielen ist. Wird in der Längsrichtung des Bauwerks ein Tunnel für den Straßenverkehr eingebracht, ist diese Wohnanlage zugleich Verkehrs- und Lärmschutzanlage für die talseitige Umgebung.
In Fig.2 ist ein beidseitiger Terrassenaufbau über einem mehrgeschossigem, steilwandigem Gebäude dargestellt. Diese Wohnanlage ist dabei herausgehoben aus der städtischen Verkehrsebene und die Wohnungen sind abgeschirmt vom Lärm der Großstadt. Solche Anlagen sind z.B. über Waren- oder Lagerhäusern, Fertigungsbetrieben oder Großraumbüros zu erstellen.
Ein vielgeschossiges Terrassenbauwerk auf einer Stützkonstruktion (Fig.3) ist auf mehrgeschossigen, seitlichen Unterbauten, die in Ortbeton auszuführen sind, herzustellen; die oberen Geschosse sind mit Hilfe einer inneren Fertigteilkonstruktion fertigzustellen. Bei diesem Gebäudekomplex ergibt sich eine Zwischenhalle, die in unmittelbarer Verbindung mit den großräumigen Erdgeschossen der seitlichen Stützbauten steht. Durch diese Halle kann der Straßenverkehr auf mehreren Fahrbahnen geleitet werden. In diesem Falle ist das Bauwerk zugleich Wohn-, Verkehrs- und Lärmschutzanlage für die städtische Umgebung. Insgesamt kann bei dieser platzsparenden, kompakten Bauweise der wertvolle städtische Boden in sechsfacher Hinsicht genutzt werden.

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A Reticulated Shell Roof for a Sports Hall in Hungary

Une toiture en treillis spatial pour une halle de sport en Hongrie

Eine Gitterschalen-Überdeckung für eine Sporthalle in Ungarn

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In 1986, ESZAKTERV, a design institute in Hungary was given the commission to design a sports hall in Nyiregyháza (a town in north-east Hungary).

The structure of the hall, briefly detailed below, covers a 60x30 = 1800 m^2 arena with grandstands in two sides and can house 3000 people.
The hall is covered by three reticulated steel barrel vaults, placed across the arena. The barrel vaults are 48 metres long with overhangs of 8 metres on both sides, spanning 15 metres. Each barrel vault with a circular profile is supported at the four corners of the 48x15 m area. The barrel vaults are supported by steel box-girders along the curved edges. The lower chords of the barrel vault are constructed in such a way that they act both as the longitudinal straight edge beams and as the starting structure of the barrel vaults.
The barrel vaults have an equilateral triangular network which consists of 2 meter long steel bars. These bars are reticulated structures themselves. The two parallel chords of these plane trusses are 0,5 metre from each other and they are connected by inner cross-bars.
According to the stress of state of the barrel vaults, the bars have three different tube cross sections. The outer diameter is 108 mm for all three types and the wall thicknesses are 10, 6.3 and 4 mm, respectively. The idea behind these different cross sections was to achieve a structure as economic as possible. The material of the tubes is easily available standard steel.
The bars in the direction of the generatrix go along the barrel vaults continuously and the other bars only go from nodal point to nodal point. The construction of the nodal points is based on an element, a tube section, which fits for the outer surface of the bars in the direction of the generatrix. This special "tube-on-tube" joint, protected by a patent, makes it possible for the joining bars to enter the joint in whatever angle is needed so the designed shape of the barrel vault can

easily be achieved by using a template. The bars are fixed by welding at the nodal points. The designed load bearing capacity of the joints were checked by tests.

To facilitate easy construction, the barrel vaults are divided into segments, in accordance with the capacity of the derrick used for the construction. These segments are prepared in advance in a construction hall. The edge girders are first put on temporary supports then the barrel vault segments are carefully placed and fixed together. Under the whole process the straight edge girders are connected by temporary steel ties of 25 mm diameter.
The construction of the hall is being carried out by ELEKTERFEM, a small company, dealing with metal structures. The sports hall will be completed by 1988.

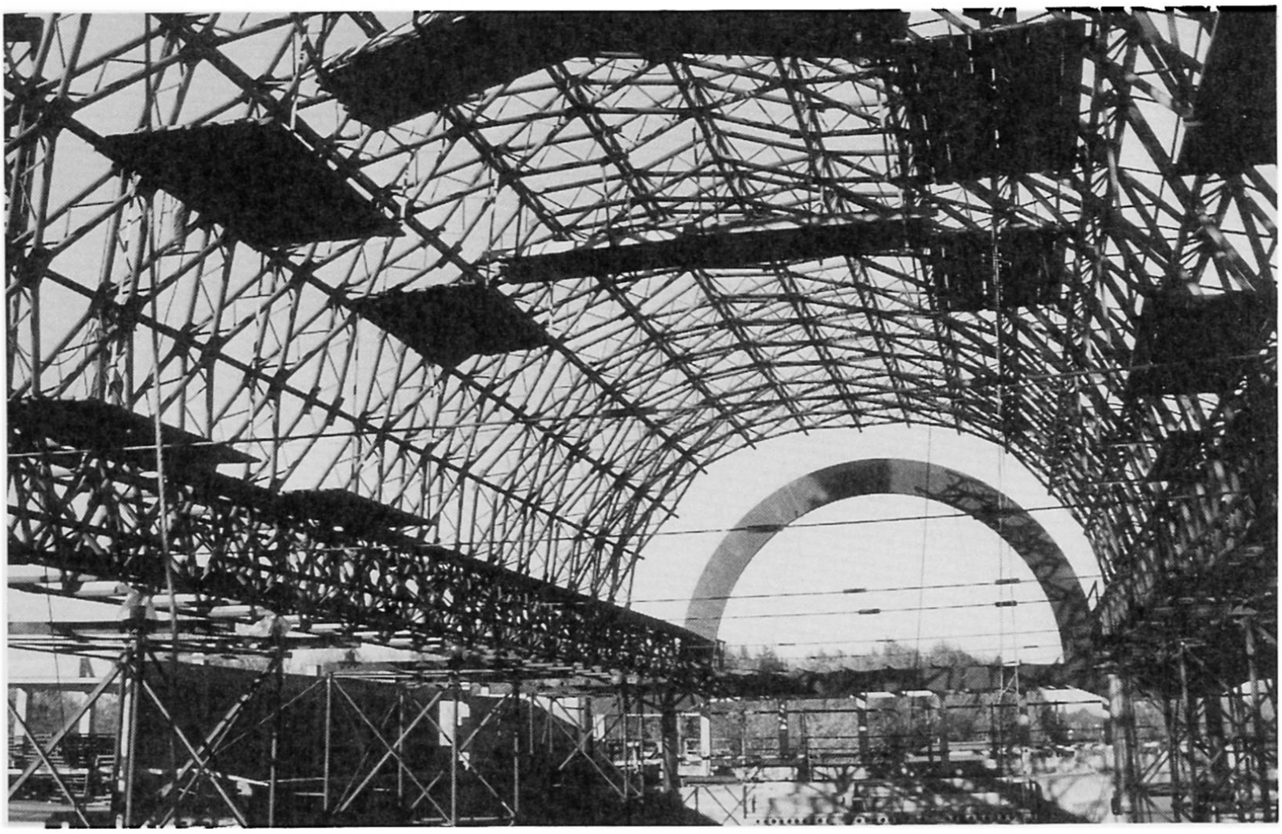

Fig. 1 Inner view

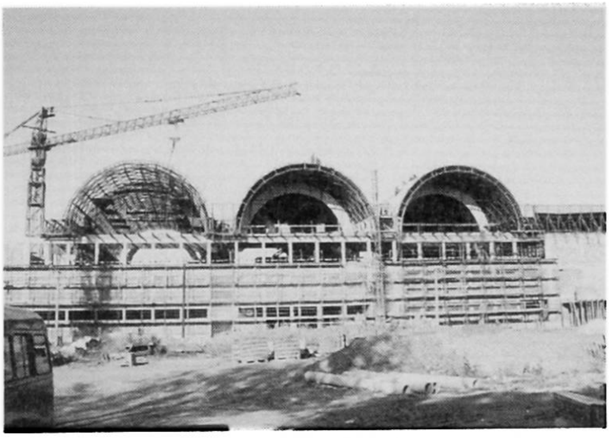

Fig. 2 General view

Fig. 3 Special tube-on-tube joint

New Truss System Using Rectangular Hollow Section

Nouveau système triangulé avec des profilés creux et rectangulaires

Neues Fachwerk-System aus Rechteck-Hohlprofilen

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1. Introduction

The rectangular hollow section steel, which is a closed section, has high torsional rigidity and shows equal sectional performance in two directions. This steel, therefore, is widely used as beam-columns members of structural frames, but the frequency of its use as long-span truss structural members is very much lower than is the case of circular hollow section steel. This is mainly because the welded joints of the chord and bracing which compose the truss are susceptible to local failures such as chord-face and chord-wall failures and punching shear. The details of welded joints are inevitably complicated to prevent such local failures, and this fact leads to fabrication complexity, structure weight increase and cost increase.

This paper proposes a new truss system (Y-truss) which has structural characteristic equivalent or superior to those of the circular hollow section steel truss, remedying the drawbacks of the rectangular hollow section steel truss and which has a nonstiffened simple joint system, reports on basic and actual-size experiments conducted to confirm the structural characteristic and shows examples of the commercial applications of this new truss system.

2. New Truss System Using Rectangular Hollow Section Steel

The newly developed rectangular hollow section steel truss system is shown in figure 1. This system has its features in the joints, as it consists in welding the chords and bracings by rotating them 45 degrees round the longitudinal axis and cutting the bracings in the shape of bird's bill. It was formerly very difficult to cut bracings in this way, but the recent development of numerical control has facilitated such cutting. This system has the following advantages. i) Since the axial loads in the bracing are transmitted as the in-plane force of the chord in the direction of 45 degrees to the axis, the flow of stress in smooth and the rigidity is high. ii) The welding length is larger than of the conventional truss system, and the strength of welded joints is higher.

3. Basic and Actual-Size Experiments on Welded Joints

Basic experiments were conducted on three types of T-joints, the new type conventional type and circular hollow section steel type, to make a comparative study of the strength of welded joints. Specimens for each having varied width ratio of bracing to chord (width of chord/width of bracing) and welding type were tested for compression and tension under static loading. For the experiments 30 specimens were used.

For both compression and tension, the welded joints of the new type truss showed higher initial rigidity and maximum load than those of the conventional type truss. The new and conventional types were compared in yielding load obtained by the general yield point method and maximum load. The yielding load of new-type truss joints was about twice that of conventional-type truss joints for a width ratio of bracing to chord of 2:1 and about 1.1 times for 3:2. The maximum load of new-type truss joints was 1.7~2.0 times that of conventional-type truss joints for a width ratio of bracing to chord of 2:1 and 1.5 times for 2:3. The difference between the two types increases as the ratio of bracing to chord increases, clearly showing the effectiveness of the new-type truss joints.

The final failure mode in tension is breaking of bracing and the mode in compression is the outward deformation of the chord local buckling in the bracing, with very little outward deformation. The new-type truss system showed stabler failure mode for both tension and compression than the conventional-type truss system. Almost no difference was noticed in deformation under load between fillet weld and butt weld. Figure 2 shows the relationship the test results which are nondimensionalized by ($\sigma_y \cdot T \cdot D$) and the width ratio of bracing to chord (d/D). The solid line in this figure are proposed a formula from the experimental results.

As shown on figure 3, actual-size experiments were carried out on high truss based on the above experimental results. New and conventional type specimens were used. As a result of the experiments, the initial rigidity, yielding strength and maximum load of the new type were found to be superior to those of the conventional type. The strength of welded joints was much higher than the yielding strength of the bracing in compression, showing that this trusses system can be designed based on the strength of truss members.

4. Examples of Commercial Applications

A gymnasium (span length 44m) and warehouse (span length 32m) were designed and constructed using this truss system. Also, an event hall (span length 108m and beam height 5.5m) has been recently designed.

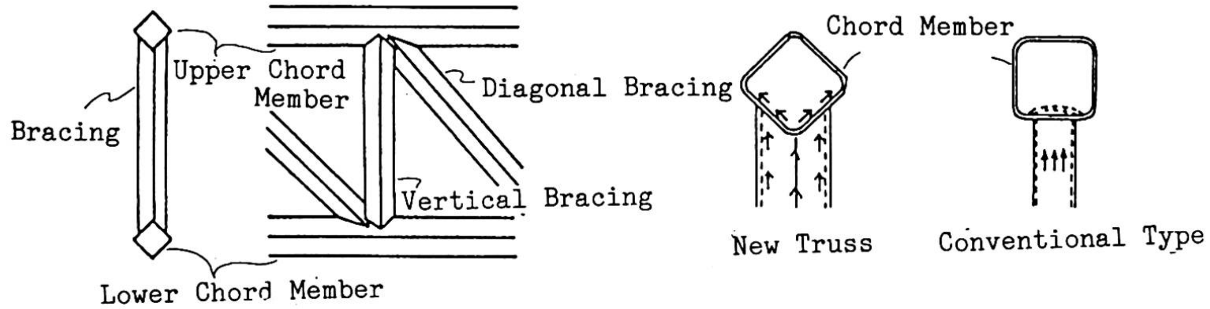
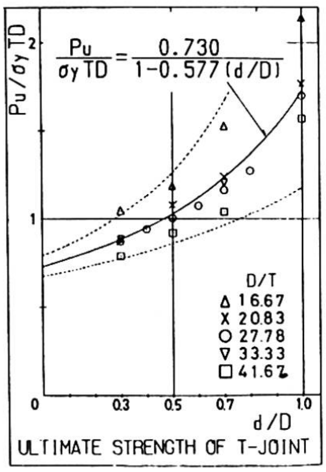


Figure 1 New Truss System



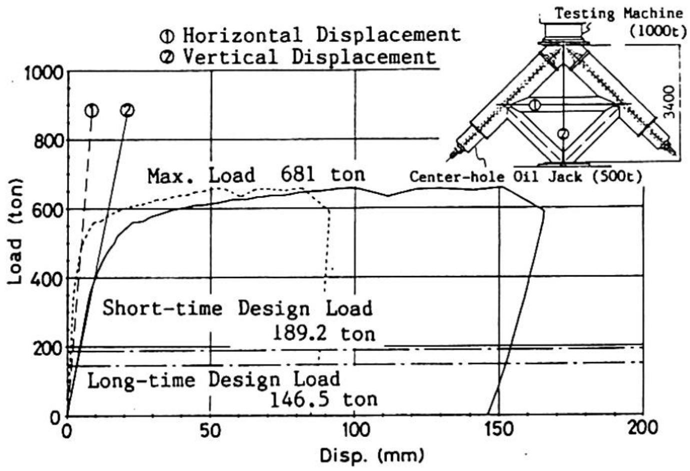


Figure 2 Ultimate Strength of T-Joint on New Truss

Figure 3 Load-Deflection Curves of Actual Size Experiments

A Castellated Hollow Section

Un profil creux à âme ajourée

Ein Wabenhohlprofil

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1. GENERAL DESCRIPTION

It is proposed to use a "castellated hollow-section" as shown in fig. 1

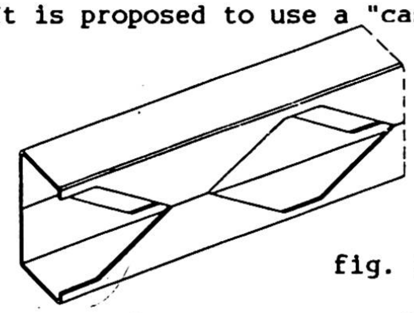


fig. 1

fig. 2

instead of the usual cold formed beams with open cross sections as shown in fig. 2.

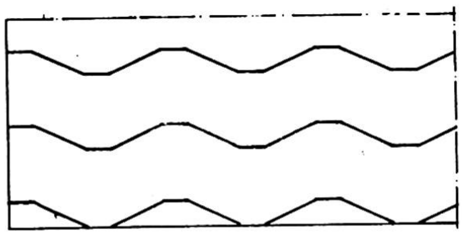


fig. 3

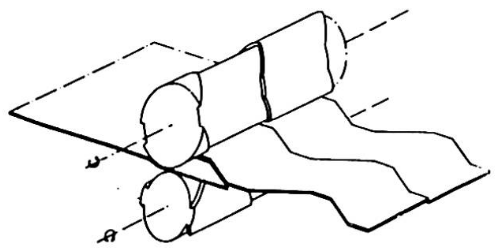


fig. 4

Such a beam is made of specially formed strips as shown in fig. 3.
These strips can be obtained out of steel sheet practically without loss of material. A proposal for a simple method is shown in fig. 4.
The beam can be produced automatized in the same way as normal welded hollow sections. So the relatively thin steel sheet on coil is the basic material. New now is the slitting with the help of specially profiled rolls as already shown in fig. 4. It is also possible to form the strips by punching.

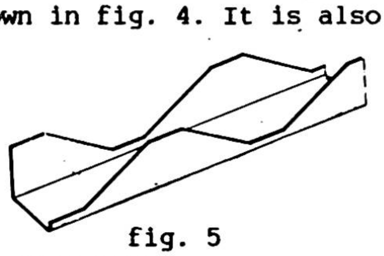


fig. 5

The strips obtained in this way can become channel shaped by cold rollings, as shown in fig. 5.
Two channel shaped parts together make one "castellated hollow section".

2. ADVANTAGES

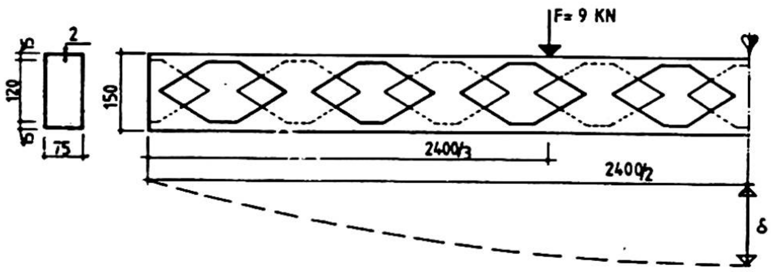
This new type of beam has the following advantages with regard to some of the usual beams:

- Great torsional rigidity.
- Economic use of material similar to that of Z. or C. profiles. Savings up to 25 - 35% are possible with regard to the normally used hollow sections and savings up to 10 - 15% with regard to the Zeta and Σ-profiles.
- A high grade of automation can be obtained by using the proposed cutting method.
- Because of the alternate position of the holes in the two webs, there are no local rather weak points as occur in normal castellated beams.
- Connections are possible by the accessibility of the interior of the beam.
- Simple application of local stiffeners against web crippling is possible.
- Far greater applicability than the usual thin walled open beams as shown in fig. 2.
- Rolled stiffeners against buckling as in the Zeta and Sigma beams are possible.
- The holes in the web allow transition of piping.

3. SOME THEORETICAL RESULTS

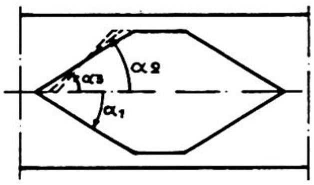
A computercalculation is made for a normal as well as for a castellated hollow section both of the same size.Hereby the castellated h.s. has 27% less material. A comparison of the results gives the following pictures

- Deflection

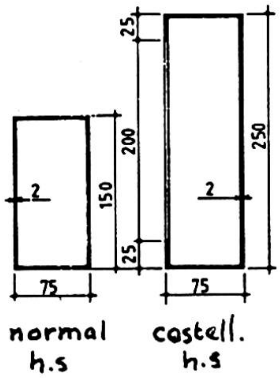


δ=7,8mm for normal h.s.
(σ_{max}=195 N/mm^2)
σ=10,2mm for castell.h.s
(σ_{max}=240 N/mm^2)

- The calculated influence on the deflection of some different hole shapes.



$tg\alpha_1$=0,6 → δ=10,2 mm
$tg\alpha_2$=0,8 → δ=10,6 mm
$tg\alpha_3$=1,0 → δ=10,9 mm



A comparison between two beams with the same use of material gives the following ratios between some characteristic values.

	normal h.s.	:	castell.h.s.
σ_{max} =	1	:	0,6
δ_{max} =	1	:	0,4
I_{tors}=	1	:	2,5

Monte Carlo Study of Strength of RC Slender Columns

Calcul de la résistance de colonnes élancées en béton armé à l'aide de la méthode de Monte Carlo

Berechnung der Tragfähigkeit schlanker Stahlbetonstützen mit der Monte Carlo Methode

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SUMMARY

The actual strength of a reinforced concrete member varies from the nominal strength which is based on specified strengths of constituent materials, geometric properties, and code design equations. The variability in the actual strength is caused by the variations in the strengths of concrete and steel, the cross section dimensions, the reinforcement placement, and the strength model itself among other factors. Computing the strength variability is an essential component in development of probability-based safety provisions for reinforced concrete design. This study was undertaken to investigate the variability of short-time ultimate strength of slender tied reinforced concrete columns of rectangular shape in cast-in-place construction. The columns studied were pin-ended with equal load eccentricities acting at both ends. The material strengths and geometric properties of the column were varied randomly and the resulting variations in the column ultimate strength were determined. The results of this study indicate that the slenderness ratio, the longitudinal steel ratio, and the end eccentricity ratio significantly influence the probability distribution properties of the column strength.

1. DESCRIPTION OF SIMULATED COLUMNS

Eighteen hypothetical rectangular tied columns subjected to single curvature bending with equal moments acting at both ends were used in this study. The graphical representation of the columns studied is shown in Fig. 1(c). Each column employed a different combination of longitudinal reinforcement ratio, specified concrete strength, and slenderness ratio (ℓ/h). All columns had the specified steel yield strength of 400 MPa, cross section size of 300 x 300 mm, concrete cover to tie reinforcement of 40 mm, and tie diameter of 10 mm. Each column was studied for end eccentricity ratios (e/h) of 0.05, 0.1, 0.2, 0.3, 0.4, 0.5, 0.6, 0.7, 0.8, 1.0, 1.5, and infinity (pure bending).

2. THEORETICAL STRENGTH MODEL

The theoretical resistance was calculated by using a strength model that generated the moment-curvature curves for different levels of axial load acting on a slender column. The maximum moment from the slender column moment-curvature curve for a given axial load level defined one point on the axial force-moment interaction diagram. This was repeated until sufficient points were obtained to define the entire interaction diagram for the slender column. The slender column resistances were then computed for specified end eccentricities through a curve-fitting subroutine applied to the generated points on the interaction diagram.

3. SIMULATION AND ANALYSIS OF DATA

Using the theoretical strength model and the probability distributions of the influencing variables described by Mirza et al. [2, 3, 4], the ultimate strengths (R) were simulated 250 times by a Monte Carlo technique for each of the end eccentricity ratios for each of the slender columns studied. The simulated strengths were then divided by the corresponding ultimate strength (R_n) predicted by the ACI Building Code [1] using the specified properties of the variables. This gave simulated samples of the ratios of the theoretical to ACI strengths (R/R_n) referred to as strength ratios in this study. All computations for the ACI strength were carried out as they would be in a design office with the exception of the strength reduction factors ϕ. The ϕ factors for the cross section strength and those for the critical buckling strength of the column were taken equal to 1.0 in this study. The results of this study indicate that the slenderness ratio, the longitudinal steel ratio, and the end eccentricity ratio significantly influence the probability distribution properties of the column strength. As expected, the variability of concrete strength is a major contributing factor to the slender column strength variability in the region of low eccentricity ratios, whereas the variability in the steel strength makes a major contribution to the slender column strength variability when the end eccentricity ratios are high. The highlights of the results obtained are summarized in Figs. 1(a), (b), and (c) which plot the range of mean and one percentile strength ratios (R/R_n) at different values of e/h for columns with slenderness ratio of 10, 20, and 30, respectively. Each of these figures represents six columns with identical slenderness ratios but different specified concrete strengths and longitudinal steel ratios. Note that each column was studied for twelve e/h ratios.

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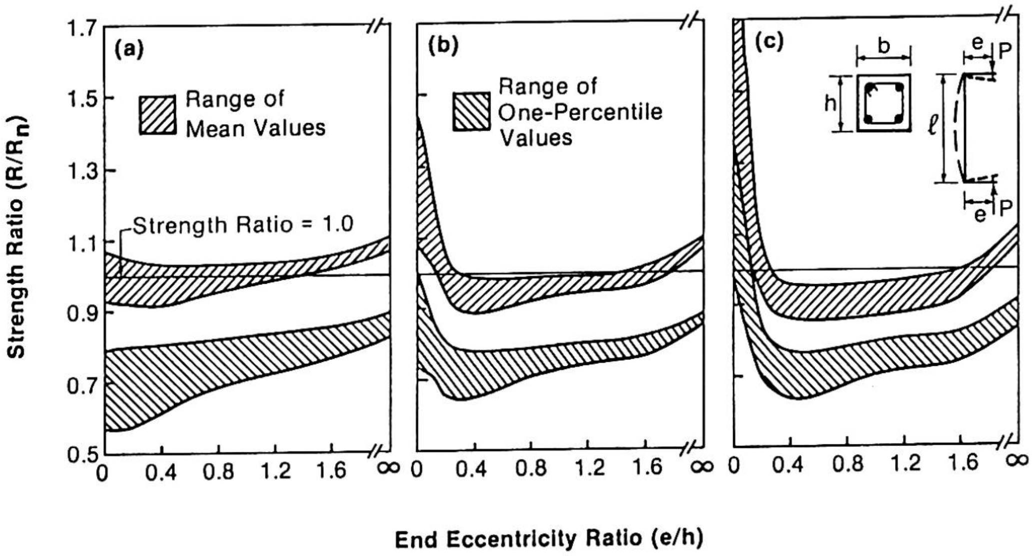

Fig. 1. Effect of end eccentricity ratio on the range of mean and one-percentile strength ratios: (a) ℓ/h = 10; (b) ℓ/h = 20; and (c) ℓ/h = 30.

Der unterspannte Träger

The cable strengthened girder

Poutre sous -tendue

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ZUSAMMENFASSUNG

Unterspannte Konstruktionen wurden bereits im 19. Jahrhundert angewendet. Der Entwicklungsstand der Baustatik ermöglichte noch keine statischen Nachweise, so daß die Anwendung dieses Konstruktionsprinzips auf reine konstruktive Überlegungen angewiesen war.
Durch die Möglichkeit, die Fachwerkträger schon frühzeitig auch mit Hilfe der Grapho-Statik berechnen zu können, wurde die Konstruktionsart des unterspannten Trägers verdrängt. Mit der Einführung der EDV wurde der große Rechenaufwand, der mit der Berechnung von unterspannten Trägern verbunden ist, erträglich. Dadurch erlebt diese Konstruktionsart eine Renaissance.

1. ZUR GESCHICHTE DER UNTERSPANNUNG ALS KONSTRUKTIONSART

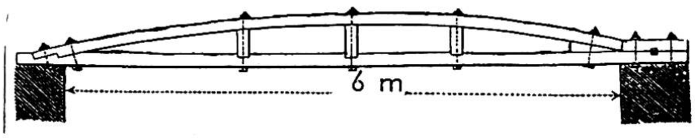


Bild 1: Gespreizter Holzbalken

Die Unterspannung als Konstruktionsart ist keine Neuentwicklung; unterspannte Konstruktionen fanden bereits Mitte des 19. Jahrhunderts verbreitete Anwendung und gehen auf den gespreizten Holzbalken zurück.

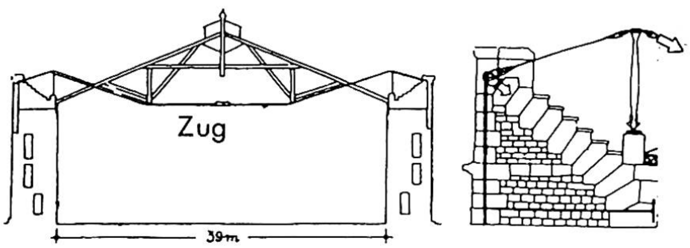


Bild 2: Überdachung in Paris nach den Plänen von Hittorff 1839

Hittorff konstruierte bereits 1839 eine unterspannte Dachkonstruktion für sein kreisförmiges Panorama-Gebäude an der Champs-Elysée

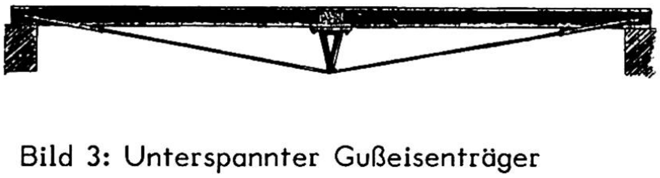
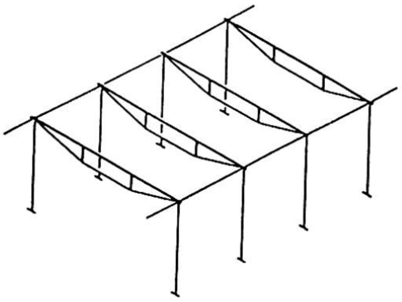


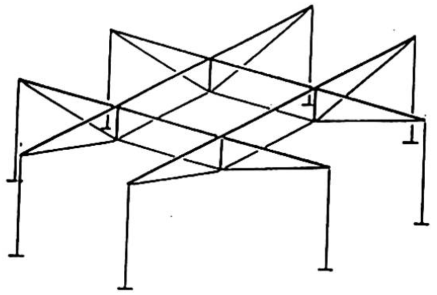
Bild 3: Unterspannter Gußeisenträger

Durch Unterspannung mit schmiedeeisernen Zugstangen wurde die Tragfähigkeit gußeiserner Träger zu Beginn des 19. Jahrhunderts verbessert.

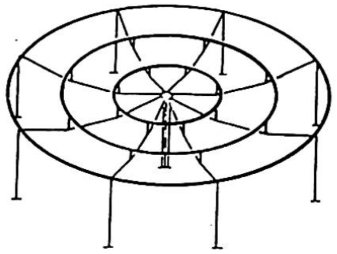
2. GRUNDFORMEN VON UNTERSPANNTEN BALKEN



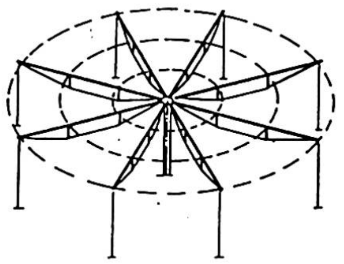
Rechteckiger Grundriß



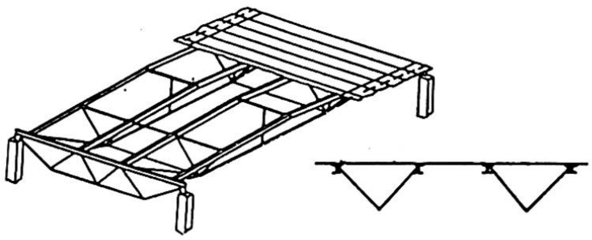
quadratischer Grundriß



kreisförmiger Grundriß mit kreisförmigen Obergurten



kreisförmiger Grundriß mit radial angeordneten Obergurten



Unterspannung mit räumlicher Tragwirkung

Die Unterspannkonstruktionen bewirken durch die aufgelöste Konstruktion gegenüber Vollwand- oder Fachwerkträgern den Eindruck einer enormen Leichtigkeit. Die Gestaltungsmöglichkeit durch die Anordnung von ein, zwei oder mehreren Steifen, abgewandelte Polonceauträger, gerichtete oder radiale Binderanordnung, ist ausgesprochen vielseitig.

Die Variationsmöglichkeiten werden aus der unterschiedlichen Anordnung der Einzelelemente, Balken (Obergurt), Spreizen und Zugbänder gebildet.
Die Obergurte können horizontal oder geneigt angeordnet werden. Auf verschiedene Grundrißformen kann man mit orthogonal oder radial gerichteter Trägeranordnung reagieren. Für großflächige Grundrisse können zweckmäßig räumlich unterspannte Konstruktionen zur Anwendung gelangen.

Durch die Dehnung der Zugbänder entstehen Nachteile im Tragverhalten des unterspannten Trägers. Dieser Nachteil kann durch die Einleitung einer Vorspannung in die Zugbänder ausgeglichen werden.

Die Vorspannung kann ohne großen Aufwand auch durch gezielte Trägerüberhöhung erreicht werden. Bei allen Unterspannkonstruktionen ist dafür zu sorgen, daß der Umlenkpunktder Unterspannung gegen seitliches Ausweichen gesichert ist. Das kann durch einen biegesteifen Anschluß der Spreize an den Obergurt, Gestaltung des Obergurtquerschnittes oder andere konstruktive Maßnahmen erfolgen.
In vielen Fällen wird es sich nicht vermeiden lassen, zusätzliche Abspannungen des unteren Spreizenpunktes anzuordnen. Hierdurch wird der gerichtete Lastabtrag in ein räumliches Tragverhalten umgewandelt. An den vorgefertigten Dachsegmenten aus der DDR ist die räumliche Tragwirkung besonders gut abzulesen.
Der unterspannte Träger stellt in statischer Hinsicht das Bindeglied zwischen den einfachen Biegebalken und dem Fachwerkträger dar. Er bietet eine brauchbare Alternative, wenn der Balken aufgrund größerer Spannweite nicht mehr wirtschaftlich eingesetzt werden kann und der Aufwand für einen Fachwerkträger zu groß ist. Mit Hilfe von EDV-Anlagen können ohne große Mühe die Einflüsse verschiedener Materialien, aus denen sich ein unterspannter Träger zusammensetzt und auch die Einwirkungen der Verformung auf die Schnittkräfte erfaßt werden.

The Roof of the Stadium in Riyadh, Saudi Arabia

La toiture du stade de Riyadh, Arabie Séoudite

Überdachung des Stadions in Riad, Saudi-Arabien

Rudolf BERGERMANN
Dipl.Ing.
Schlaich und Partner
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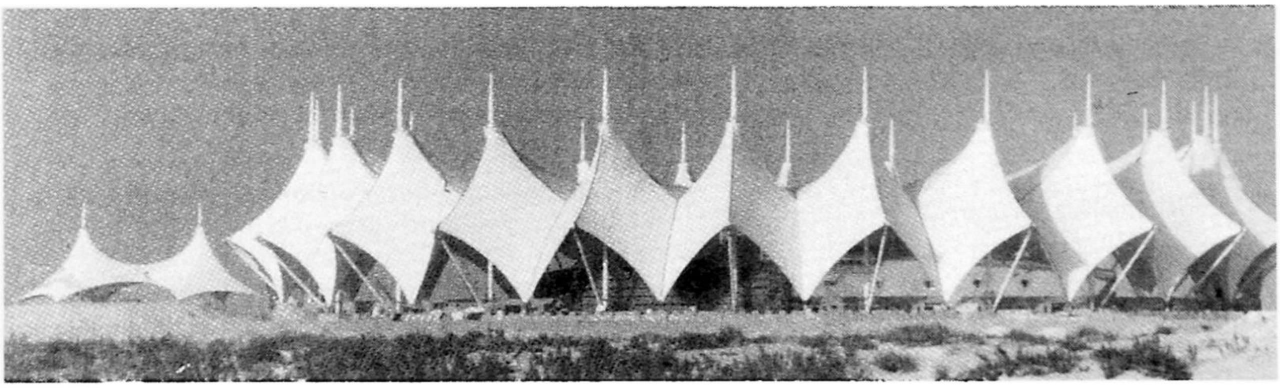


Fig. 1: Riyadh Stadium Roof

Following a design of J. Fraser, J. Roberts and Partners, London, with Geiger and Berger, New York, for the Ministry of Youth and Welfare, Riyadh, as clients, the grandstands of the new stadium in Riyadh are covered by a 50,000 m^2 membrane roof to protect visitors against sun and rain. Each of the 24 units, arranged in an annular shape of 134 m inner and 270 m outer diameter, consists of:

- a vertical main mast
- a pair of suspension and stabilizing cables, which are put under prestress by the center ring cable,
- the staying system, comprising an upper stay and two triangulated lower stay cables, deviated by a 45° inclined edge mast.

These three elements form a stabilized primary system. The membrane units, edged by ridge, valley and catenary cables, are attached to the main mast top ring, the top of the edge mast, the center ring knots and directly to foundations as secondary elements and stressed in between those anchor points.

The masts, cylindrical steel pipes of plates up to 30 mm thickness and with conical end parts center their loads on hinge bearings which are provided mainly to cater for rotational movements during construction. To ease transport, the main masts got 2 HSFG-bolt splices.

All cables are of a locked coil type with up to 3 layers of Z-shaped wires. The diameter variation between 26 and 74 mm yields ultimate cable loads of 650 to 5,100 kN. All wires are hot dip galvanized. The outside corrosion protection of cables and sockets is guaranteed by a 2 mm polyurethane coating, which shows a good bond on galvanized surfaces, high ductility up to 400 %, high tensile strength and an excellent durability against sand abrasion and aging due to UV-radiation.

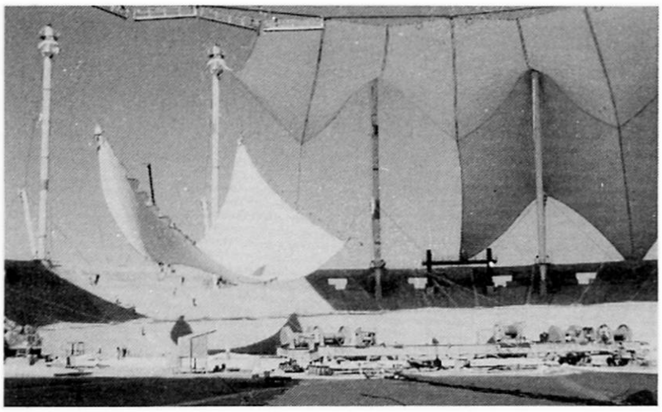


Fig. 2: The membranes are lifted

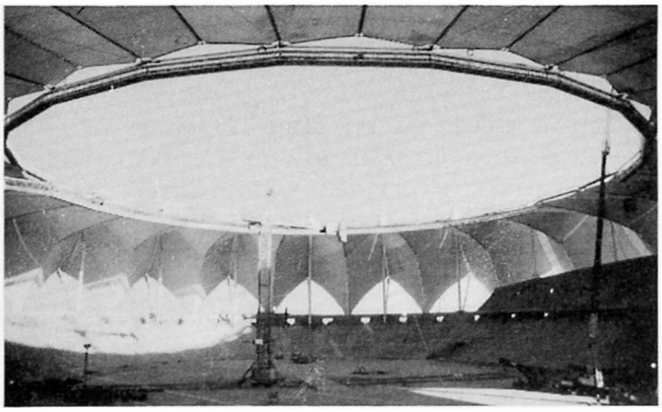


Fig. 3: The roof seen from inside

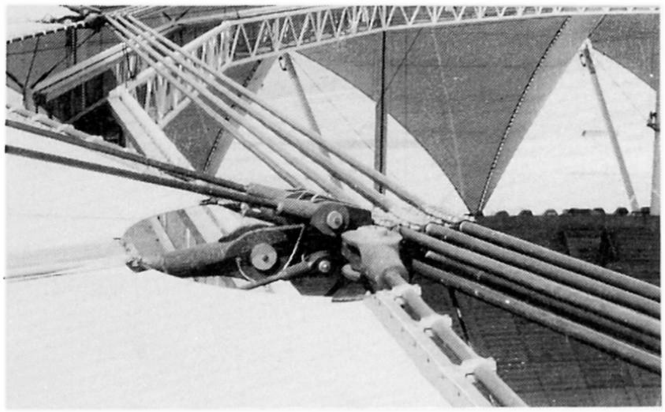


Fig. 4: Center ring detail

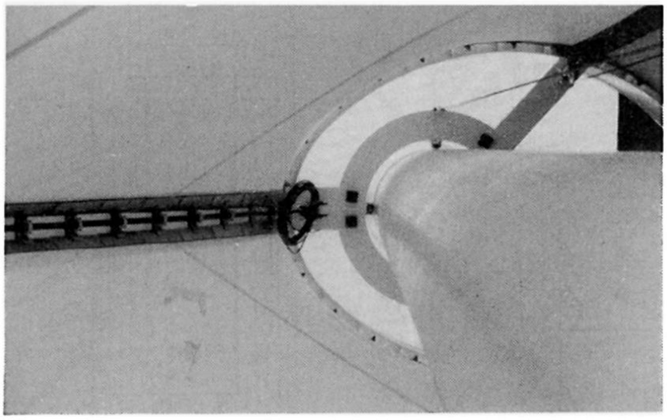


Fig. 5: View of mast top from inside

The membrane is a PTFE-coated glassfibre fabric; its ultimate strength is specified as 150 kN/m in both directions in order to assure a safety factor of >10 for permanent loads like prestress and >5 for short time loads like wind. Each roof unit consists of 4 individual membrane parts, resulting in a maximum size of 850 m^2 per piece; these were manufactured in the shops out of 4 m wide strips, according to a predetermined pattern and folded into a crate for transport to site.

The construction started with the erection of the primary system; the center ring cable, laid out and assembled on ground, was pulled straight up into position by the suspension cables and jacks on the 24 mast tops; the rear stay cable system was lifted piece by piece by a mobile crane. After addition of temporary guy cables from the center ring down to the ground the primary system was slightly prestressed for stabilization.

The membrane units were pre-assembled on the ground including catenaries and their attachment knots. Then they were trolleyed by a system of guide and pulling ropes into their final position, anchored to the primary system and stabilized by ~ 20 % prestress. The final prestress was applied after installation of all fabric elements by jacking the main mast rings up. The specified membrane prestress of 40 pli was reached with a tolerance of a few centimeters only for the ring levels.

General contractors for the stadium were Holzmann AG, Frankfurt, with Chemfab, Birdair Structures Group, Buffalo, N.Y., as subcontractors for the roof. The detailed design of the membrane, cable and steel structure including the technical site supervision was, in close cooperation with Birdair, Buffalo, entrusted to Schlaich und Partner, Stuttgart.

Control of Indoor Climate by an Integrated HVAC and Building System

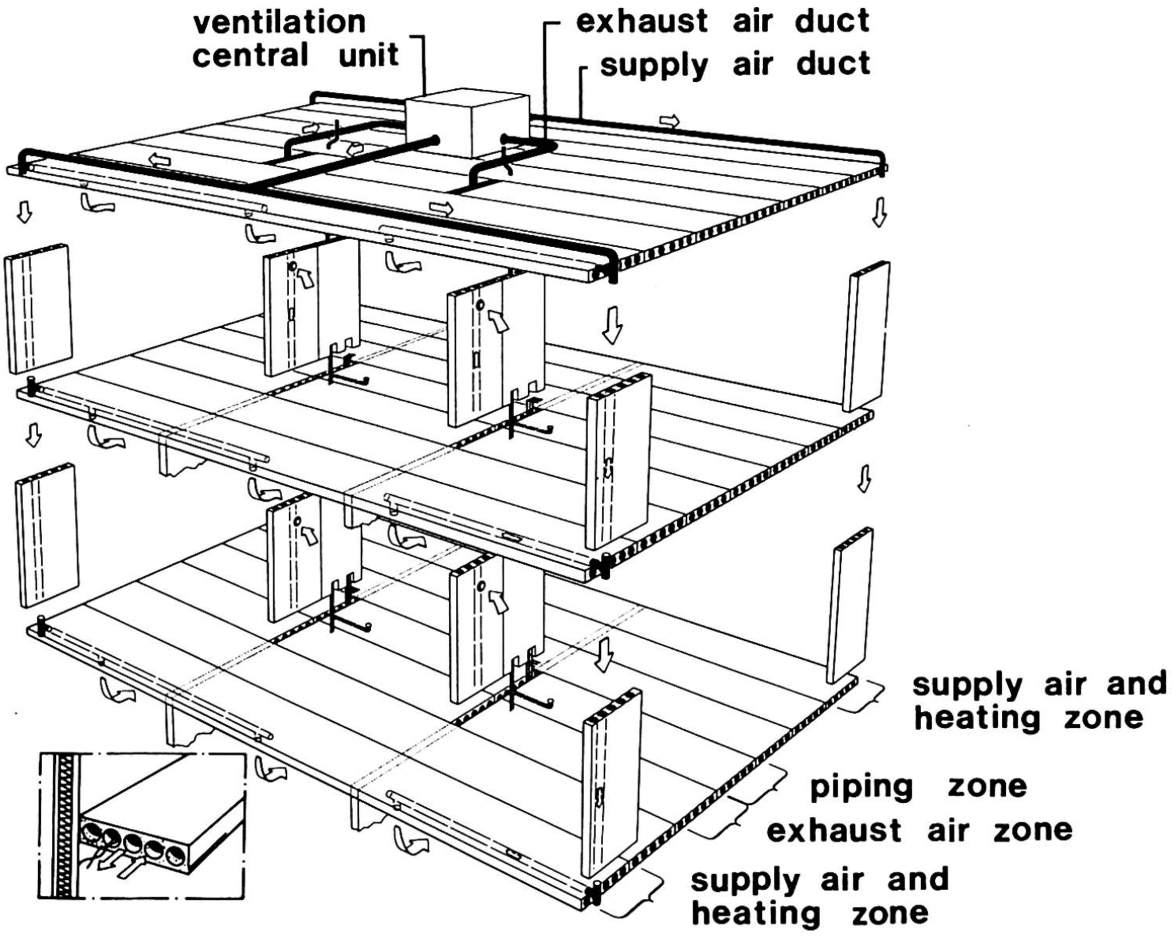
Contrôle du climat intérieur par un système intégré de climatisation et construction

Innenklimaregelung mit einem integriertem Klima- und Bausystem

Juhani LAINE
Senior Research Scientist
Technical Research Centre of Finland
Espoo, Finland

EBES - integrated HVAC and building system

The heating and ventilation systems and structures in an open Finnish BES-element building system of multi-storey residential buildings are integrated into an energy economic overall EBES-system. The mechanical sound attenuated supply and exhaust ventilation system is equipped with heat recovery.



EBES - indoor climate and ventilation

The objective values for EBES-apartment indoor climate meet the recommended values in the new compiled Finnish Building Regulations D2:1987. In addition the following most important requirements of flexibility for man are fulfilled:

• Ideal room-based controlled temperature	21 ± 2	°C
• Controlled ventilation	0,65 - 1,0	1/h
• Draughtless silent ventilation	25 - 30	dB(A)
• Prevention of spreading smells		
• Good sound insulation		

EBES - air ductwork

The building frame in the EBES-system comprises floors made of hollow core slabs and bearing walls of hollow core elements. The hollows in the structures are used as supply and exhaust air ducts. The air ductwork is simple to design, easy to adjust, flow-technically stabile, sound technically controlled and allows changes in air flows when:

Maximum air velocities in the ductwork	m/s
• Main ducts of sheet metal	4,0
• Mutual ducts (vertical hollows)	3,0
• Room ducts (vertical or horizontal hollows)	2,0

EBES - building frame as energy storage

The exploitation of the building mass for storage of energy and as a part of the heating and ventilation system is profitable when combined radiation and ventilation heating is used. The building mass can be heat accumulated by preheating the supply air led into the hollow core structures or by installing electrical heating cables into the hollows. A room-based air terminal device heats the supply air and controls the indoor air temperature.

EBES - functional prerequisities

To avoid the disturbances that change the air flows in the ducts and in the whole building causing for example condensation and spreading smells and are caused by the thermal forces due to the temperature difference between the outdoor and indoor air and by the wind, the functional prerequisities in the cold, t < -20 °C, climate for EBES-system are:

• Maximum leakage air flow rates at test pressure of	50	Pa
Outer building envelope	0,5	1/h
Floors	0,1	$dm^3/(sm^2)$
Doors	2,0	dm^3/s
Air ducts	0,04	$dm^3/(sm^2)$
• Minimum static pressure in the air ducts	100	Pa
• Mechanical supply/exhaust air flow rate	0,8	
• Continuous air change		
• Sound attenuated air terminal devices		
• Control technique for the room-based heating effects.		

Foundation Developments for Arctic and Poor Soils Conditions

Nouveau type de fondations pour des conditions de sol difficiles

Neuartige Fundationen für arktische und wenig tragfähige Böden

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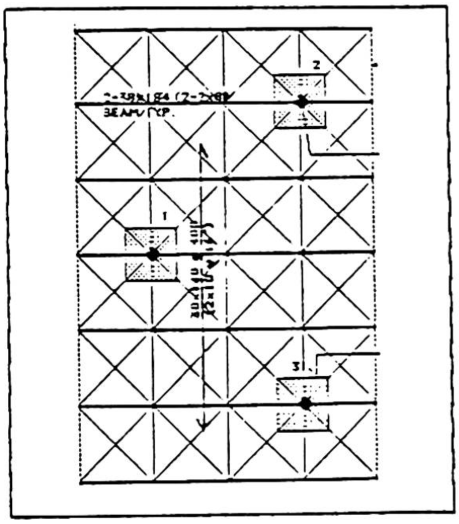
Canada Mortgage and Housing Corp.
Ottawa, Canada

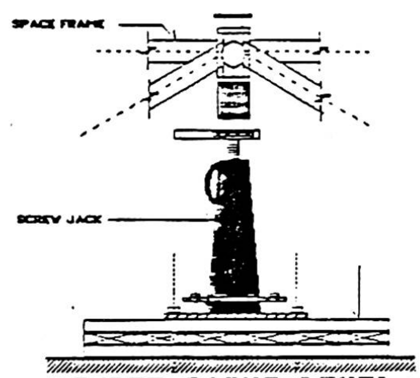
It is expensive to repair or replace residential buildings in remote Arctic locations damaged by differential foundation movement and frame racking. This paper highlights a Canadian Arctic residential foundation research program directed toward the development of practical and cost effective structural systems that will eliminate this problem. A tubular metal space frame chassis and plywood monocoque superstructure on three point or multiple point bearing conditions are two systems that are presently undergoing both theoretical computer modelling as well as field demonstration and testing.

Various combinations of geotechnical and structural foundation systems are traditionally used in Arctic residental construction including piles, surface pads/wedges and buried pad and pier. The performance of these systems are dependent on the construction process, geology, and level of maintenance. A purely structural solution reduces the number and complexity of the variables and increases the probability of providing long term stable support for the residential structure. A three dimensional metal space frame under the house or a stress skin shell structure encorporated into the wood frame superstructure are the two structural systems being developed. Either structural system could be simply supported on three determinant bearing points which has the advantage of eliminating torsional racking forces but increasing bending and bearing stresses.



THREE POINT SPACE FRAME MODULE PLAN

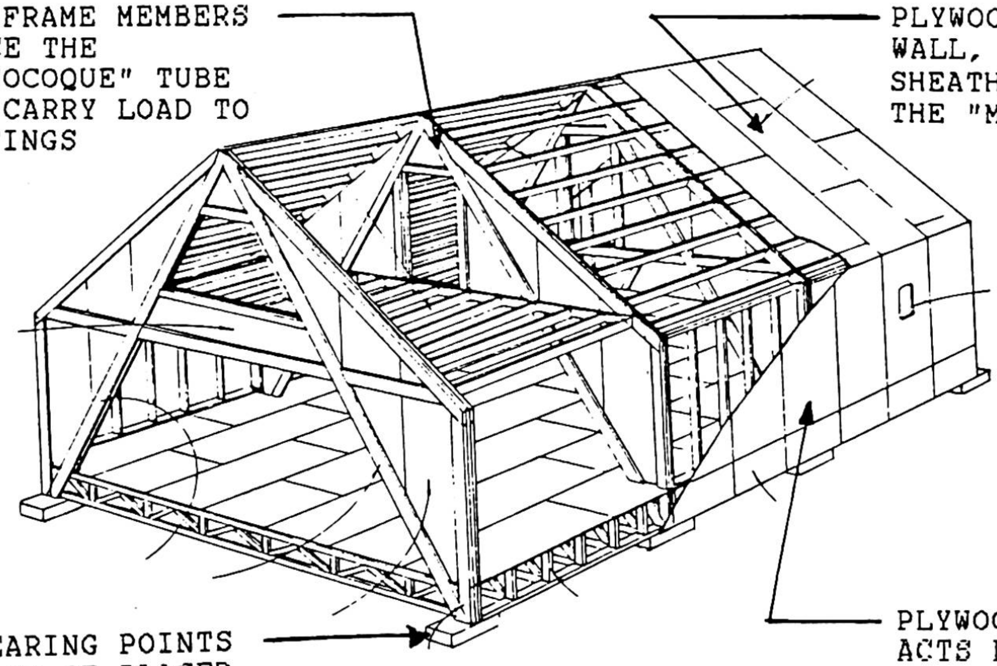
A space frame on three bearing points was designed and manufactured by Triodetic Building Products Ltd. and constructed under a house in Rankin Inlet, NWT in 1985, whose steel pile foundations had failed. Maximum deflections were calculated to be 44 mm but measured to be 7 mm with no damage to the plaster board finishes. Double tubing was required at each of the three bearing points to reduce the stresses caused by the focus of loads. Other disadvantages of the three bearing point concept are that the structural system limits the floor plan and specialized engineering design and construction are required.



THREE POINT LEVEL SCREW JACK DEVICE

A modular multiple bearing point concept would overcome the three point limitations but would also introduce torsional racking forces. Both the space-frame and monocoque systems can be designed to resist torsional racking forces caused by differential ground displacement however, to reduce the structural framing requirements, it is advantageous to dampen footing displacements. Proto-type development and testing of dampening devices such as metal springs, air bags, and foam gasket materials are underway. The top of the soil layer may provide a dampening effect and reduce over-stressing forces by slighlty undersizing the footing plates so that any overloaded footing will fail the soil on a local basis and redistribute the load. A frame was installed in Hay River, NWT, in 1987 that is designed according to this philosophy.

A multiple bearing point and rack resistant monocoque structure is in the design stage and will be constructed and lab tested in Vancouver, B.C. in 1988. This full scale testing under controlled loading and displacement conditions will produce optimum designs for production purposes. Although these foundation systems are being developed in Arctic soils conditions such as discontinuous permafrost, the concepts could find application in conditions where soils are generally unstable such as expanding clays and unstable silts.



MONOCOQUE CONCEPT

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