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I

Les idées de base dans la conception des structures et le choix des solutions possibles

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I a

L'influence des moyens et des méthodes de construction

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Planning of Structures and its Relationship with Construction Methods

Amtssitzgebäude der UN-City in Wien
Einfluss der Baumethoden auf den Entwurf von Tragwerken

Headquarters Building of UN-City, Vienna
Influence of Construction Methods on the Design of Structures

Bâtiments du siège de l'ONU à Vienne
Influence des méthodes de construction sur le projet des structures

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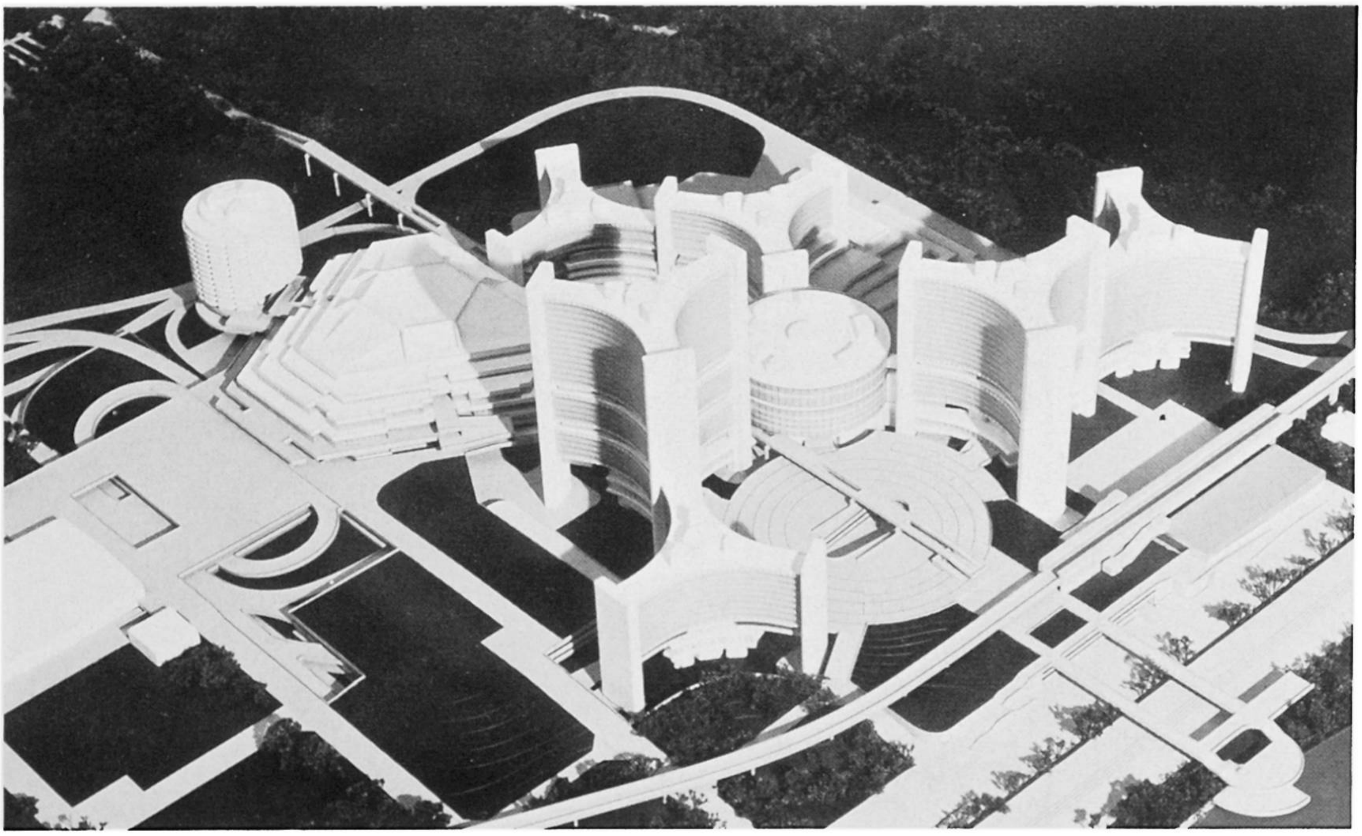
1. Einleitung und Aufgabenstellung

In der ersten Baustufe der als UN-City bezeichneten Anlage nehmen die dem Bürobetrieb dienenden Amtssitzgebäude eine dominante Stellung ein. Nach dem Entwurf des österreichischen Architekten Johann Staber werden für 4700 Personen vier im Grundriß gleiche, Y-förmige Gebäude mit Höhen von 56 m - 117 m errichtet. Da der Raum unter den Gebäuden weitgehend frei für Verkehrs- und Nebeneinrichtungen bleibt, beginnen die eigentlichen Bürogeschoße erst in 28 m Höhe und erfolgt die Abtragung der Gebäudelasten über wenige, in der Mitte und an den Enden der Y angeordnete Stützungselemente, die durch brückenartige Spannbetonträger verbunden sind. Auf diesen ruhen die Tragskelette der Regelgeschoße.



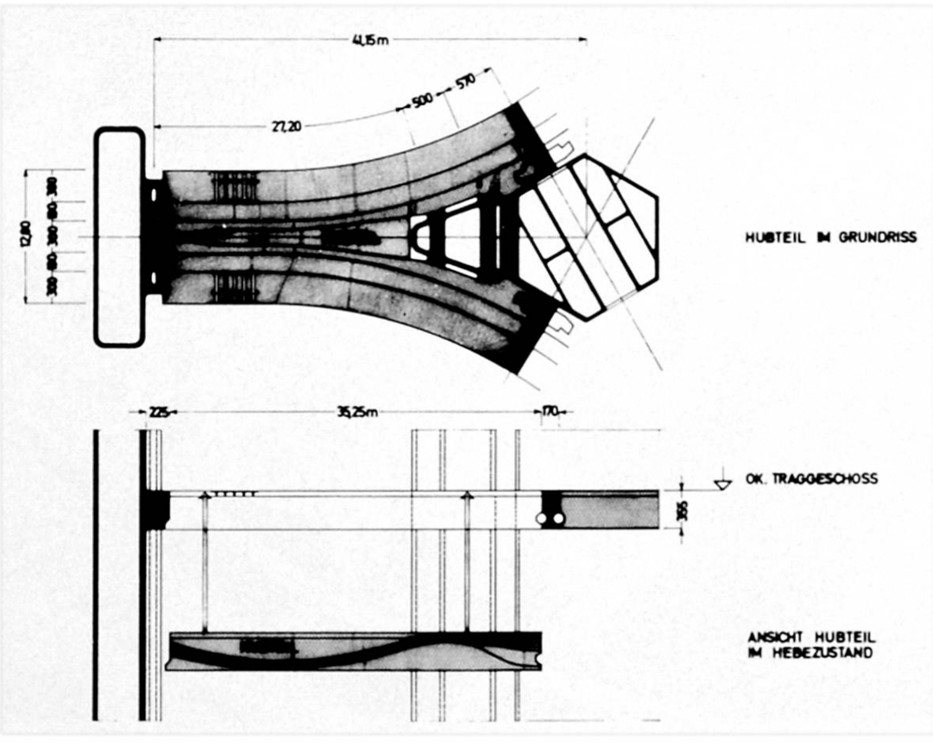
Die Planung dieser Objekte, die sich im Grundriß, in der Höhenentwicklung und im Querschnitt stark von üblichen Hochhausbauten unterscheiden, war gekennzeichnet durch einige neuzeitliche Ausführungsmethoden, die mit der besonderen Architektur starken Einfluß auf die Konstruktion ausübten.

Für den statischen Entwurf ergaben sich somit folgende allgemeine Vorbedingungen:

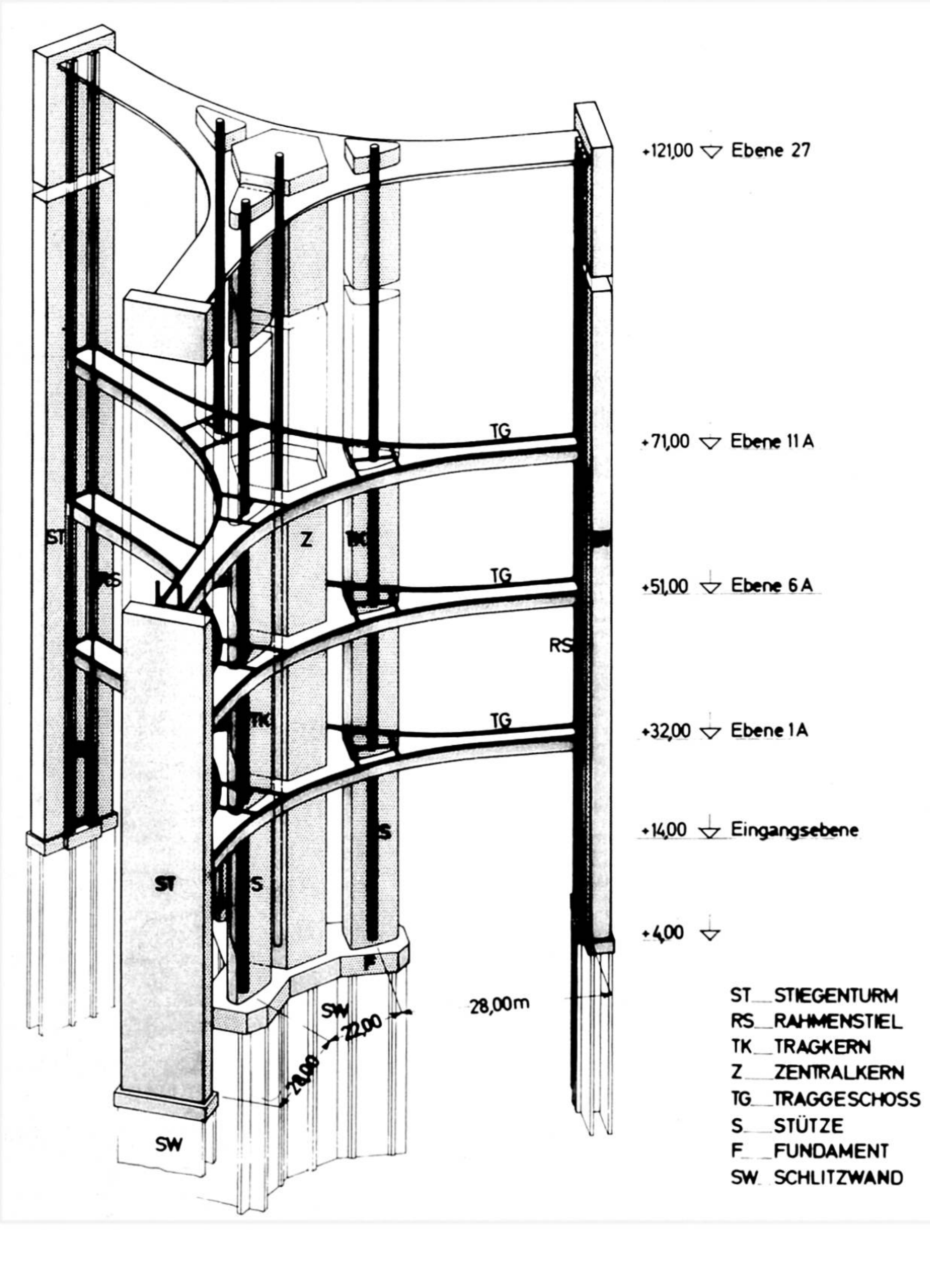
- Errichtung und Vorziehen aller lotrechten Tragglieder in Gleitbauweise.
- Gerüstlose Herstellung der die Gebäudemassen aufnehmenden Traggeschoße als dreiteilige Großfertigteile am Boden und hydraulisches Heben derselben in die endgültige Lage im Bauwerk (Hubgewicht der Einzelteile 1400 t).
- Ausbau der Regelgeschoße unter Verwendung möglichst vieler und gleichartiger Fertigteile.
- Ableitung der in Geländehöhe konzentriert anfallenden Gebäudelasten in tiefere Bodenzonen, da an der Oberfläche des Baugeländes nur heterogene und kompressible Schichten anstehen.

2. Haupttragkonstruktionen

Eine der Hauptaufgaben der Planung bestand nun darin, die Hubteile der Traggeschoße kraftschlüssig an die vorgezogenen Gleitkonstruktionen anzubinden. Beim höchsten Objekt sind drei solcher Traggeschoße angeordnet, die Einbauhöhen befinden sich in 28,48 m und 67 m über Gelände, die freie Spannweite beträgt bis zu 30 m. Der schwerste dieser brückenartigen Träger liefert im Extremfall an der Außenstützung rd. 2600 t und im Gebäudeinneren, bei den ovalen Mittelstützen, über 6000 t Auflagerkräfte. Die Überleitung dieser Kräfte von den Traggeschoßen in die lotrechten Wände gelang über indirekte Lagerung mit Hilfe von Auflagerquerträgern, die nachträglich in Aussparungen der Gleitwände betoniert und mit den Stegen der Traggeschoße spannbetonmäßig verbunden werden. Durch die waagrecht und schräg geführten Spannglieder wird die in der lotrechten Kontaktfuge zwischen Hubteil und Querträgerortbeton wirksame Querkraft reduziert und eine so große horizontale Druckkomponente erzeugt, daß die Sicherheit gegen Abgleiten des Hubteiles allein durch den Reibungsschluß gewährleistet werden kann. Im nachträglich betonierten Querträger können die durch die Hüllrohre des Fertigteiles eingezogenen Spannglieder leicht ausgerichtet werden. Die unvermeidlichen Ungenauigkeiten der Gleitbauweise wirken sich dabei nicht erschwerend aus.



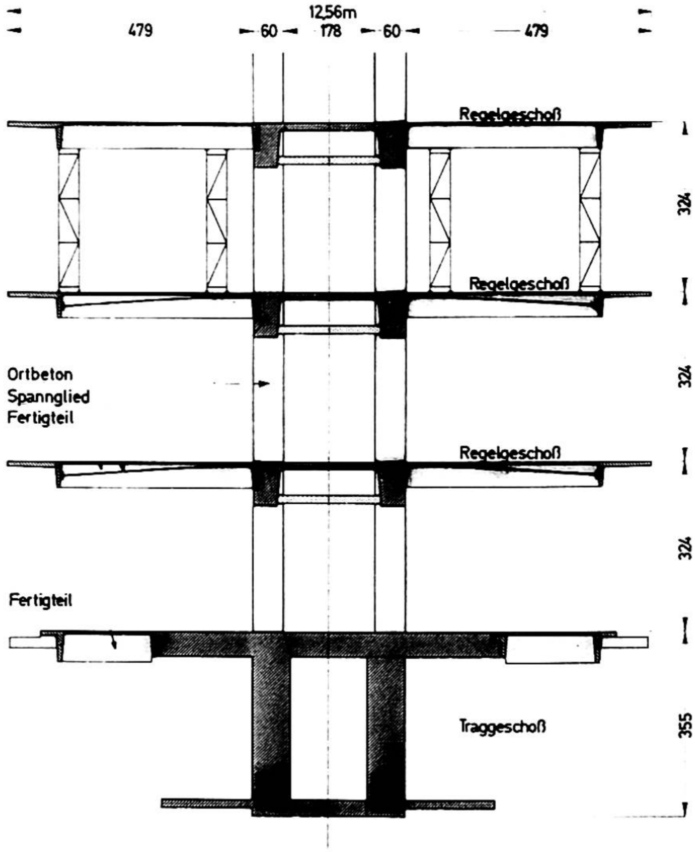
Von ganz wesentlicher Bedeutung war weiters die Erzielung eines genügend steifen Systems zur Ableitung der Seitenkräfte infolge Windwirkung und etwaiger Erdbebenkräfte, bedingt durch die Lage des Baugeländes an den Ausläufern eines Erdbebengebietes. Die naheliegende Lösung mittels kräftiger Scheiben hätte aber Einschränkungen in Funktion und Architektur gebracht. Es wurden daher die im Grundriß aus viereckigen und ovalen Röhren bestehenden Stützen mit den vorgespannten Traggeschoßen zu einem räumlichen Rahmensystem verbunden, welches durch den kräftigen sechseckigen, vom Fundament auskragenden Zentralkern in Gebäudemitte noch zusätzlich versteift wird. Die an den äußeren Enden der Objekte befindlichen Stiegentürme sind zur Vermeidung größerer Zwängungen vom Rahmen getrennt, durch Schubleisten und Verhängungen werden aber ihre längeren Scheiben am Rahmensystem zur Mitwirkung gebracht. Unter Vernachlässigung des auf den Traggeschoßen stehenden Skelettes der Regelgeschoße existiert somit ein allseits verschiebliches, räumliches Rahmensystem, das sich infolge der großen Basis und der kräftigen Einzelelemente günstig zur Ableitung der Seitenkräfte eignet. Beim höchsten Objekt weist dieses System etwa 1200 statisch überzählige Größen auf.

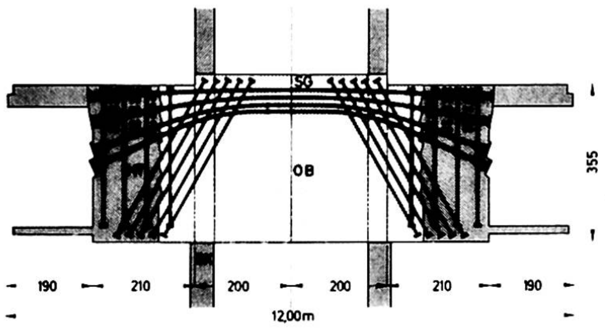


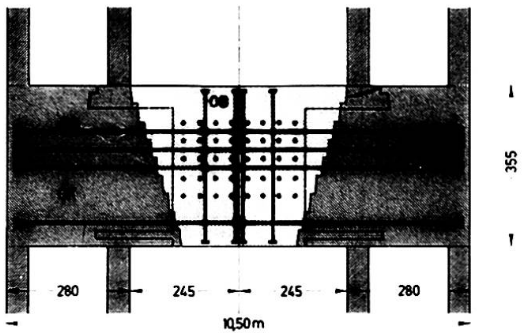
3. Einige konstruktive Besonderheiten

Das Skelett der Regelgeschosse weist an der Außenfront nur eine Metallfassade ohne tragende Stützen auf. Die Decken bestehen aus TT-förmigen Fertigteilen, welche von den Mittelunterzügen 4,80 m frei auskragen und durch Spannglieder mit dem zeitlich vorlaufend hergestellten Ortbetonskelett verbunden werden. Die Weiterleitung der Kragmomente erfolgt über die Gangplatten im Verein mit waagrechten Aussteifungen an der Unterseite der Unterzüge.

Die nachstehenden Bilder zeigen das Prinzip der Verbindung der schweren Traggeschoße mit dem Tragkern und die Einbindung der Traggeschoße in die Endquerträger der äußeren Rahmenstiele.





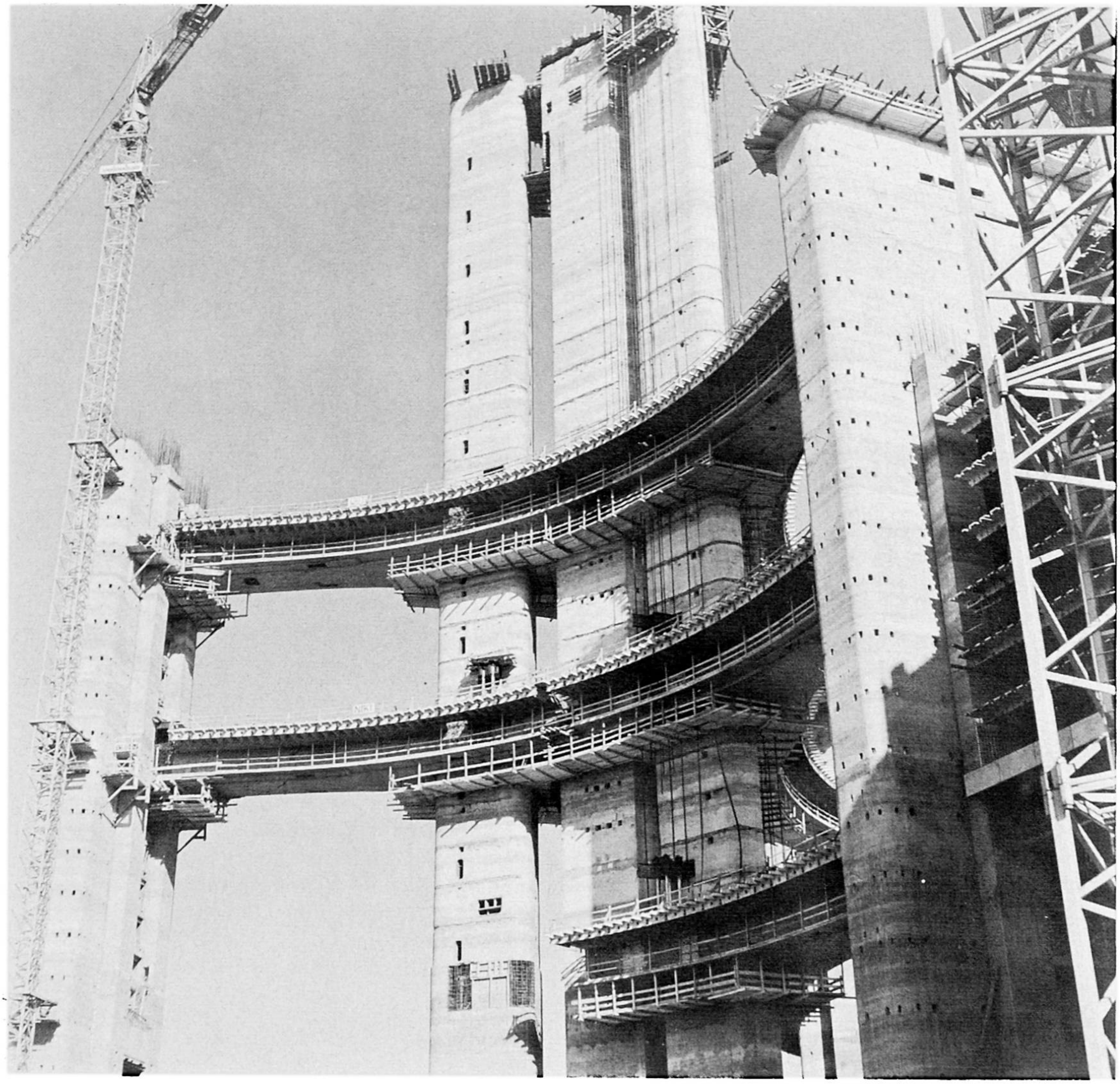


GK GLEITKONSTRUKTION
OB ORTBETON
HT HUBTEIL
SG SPANNGLIED

Da die relativ steifen Haupttraggeschoße empfindlich auf mögliche Setzungen reagieren, werden die äußeren Rahmenstiele nicht mit dem Fundament fest verbunden und so ausgebildet, daß ein späteres Heben oder Senken mit Hilfe hydraulischer Pressen vorgenommen werden kann.

4. Gründung

Die Ableitung der Gebäudelasten erfolgt auf direktem Wege mittels Schlitzwandgruppen, die einheitlich 24 m tief in die fester gelagerten Zonen des Baugrundes reichen. Für die Einleitung der Lasten in diese Schlitzwände werden in Gebäudemitte und an den Enden voneinander getrennte Plattenroste vorgesehen. Das Trag- und Setzungsverhalten dieser Gründungskörper wurde vor Beginn der Detailplanung an einem Baustellenversuch studiert. Die laufenden Messungen zeigen Setzungen in dem von den Gutachtern prognostizierten Bereich.



5. Baudurchführung

- Herstellung der Stiegentürme, Rahmenstiele, Tragkerne und des Zentralkernes in Gleitbauweise.
- Herstellung des oberen Traggeschoßes in drei Teilen (zeitlich verschoben) am Boden und Vorspannen einiger Spannglieder.
- Heben des ersten Traggeschoßdrittels mittels hydraulischer Pressen, die im Bereich des Stiegenturmes und des Tragskelettes situiert sind (Geschwindigkeit ca. 5,0 m je Tag). Abhängen des ersten Drittels und Anheben der zwei weiteren Teile. Betonieren der Auflagerquerträger und Vorspannen weiterer Spannglieder.
- Wiederholung des Vorganges für die tieferliegenden Traggeschoße, sodaß die Traggeschoße nun überall mit den Stützen biegesteif verbunden sind.
- Herstellung des Ortbetonskelettes der Regelgeschoße gleichzeitig auf allen Traggeschoßen.
- Versetzen der vorgefertigten Kragdecken und Verbindung mit Ortbeton durch Vorspannung. Während der beiden letztgenannten Bauphasen werden die restlichen Spannglieder gespannt.

ZUSAMMENFASSUNG

Bei der Errichtung der UN-Amtssitzgebäude in Wien wurden zuerst die in der Mitte und an den Enden gelegenen lotrechten Tragglieder im Gleitverfahren errichtet. Dann wurden die verbindenden brückenartigen Hauptträger am Boden hergestellt, hydraulisch gehoben, mit den Gleitkonstruktionen verbunden und darauf das Tragskelett der Regelgeschosse aus Ortbeton und Fertigteilen aufgesetzt. Diese neuartige Bauweise erfordert viele neue konstruktive Lösungen, unter weitgehender Verwendung von Spannbeton.

SUMMARY

Erection of the UN office towers in Vienna was realized in several steps: first the hollow columns, situated at the ends and in the center were concreted using sliding formwork, then the main girders were fabricated on ground level and afterwards lifted hydraulically in their final position. There, they were connected with the hollow columns by means of prestressing tendons and concrete cast in place. There upon the structure of the upper floors was erected, combined of precast elements and cast in place concrete. The usual load carrying system and the erection method challenged for many new structural details with extensive use of prestressed concrete.

RESUME

Lors de la réalisation des bâtiments administratifs de l'ONU, à Vienne, on a tout d'abord érigé par coffrage glissant les cages verticales au centre et aux extrémités. Puis on a construit les éléments porteurs principaux au sol, on les a élevé hydrauliquement et assemblé aux structures verticales. On a mis en place le squelette porteur des étages, réalisés en béton armé préfabriqué et coulé sur place. Cette méthode de construction inhabituelle a exigé de nouvelles solutions constructives utilisant le béton précontraint.

Flexible Composite Construction Types for Urban Expressways

Souplesse des constructions mixtes pour les routes urbaines surélevées

Flexible Verbundlösungen für Hochstrassen

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1 - INTRODUCTION

The construction of modern expressways in large urban areas pose some important questions on the structural solution and design.

In order to solve traffic flow problems in densely constructed and populated areas, elevated expressways must be built through existing streets or avenues and partly through expropriated areas.

The basic requirement for such a construction, besides the one of keeping costs as low as possible on account of heavy public investments, is that it must not interfere with the normal flow of traffic.

Most frequently, changes of design must be made during the process of construction, on account of unpredictable hindrances, such as existing pipelines, old foundations and so on.

Sometimes, the structural solution must be flexible enough, in order to allow removal and use in other sites. Normaly, provisional supporting structures and scafolding are not compatible with local traffic requirements.

The above conditions are determinant for the choice of the structural solution.

The characteristics for the adequacy of a solution, in order to comply with the above requirements,is that:

a) the structural solution be flexible enough, in order to allow a local change of design, without affecting considerably the overall conception;
b) the structural solution rests mainly on independent and possibly self-supporting constitutive elements.

We shall in the sequel outline a solution, which has been proposed for expressways through Rio de Janeiro and São Paulo, Brazil.

This line of solution has been adopted, in many instances, in Rio de Janeiro, where it is under construction.

2 - FLEXIBLE COMPOSITE SOLUTION

A flexible type of composite construction, in order to comply with the previous requirements could be designed with the following main features:

1) a composite superstructure, with steel beams and precast or cast "in situ" roadway slab;
2) prestressed concrete traverses, in order to convey the loads from the superstructure to the columns;
3) reinforced concrete columns.

The solution is illustrated in Fig. 1

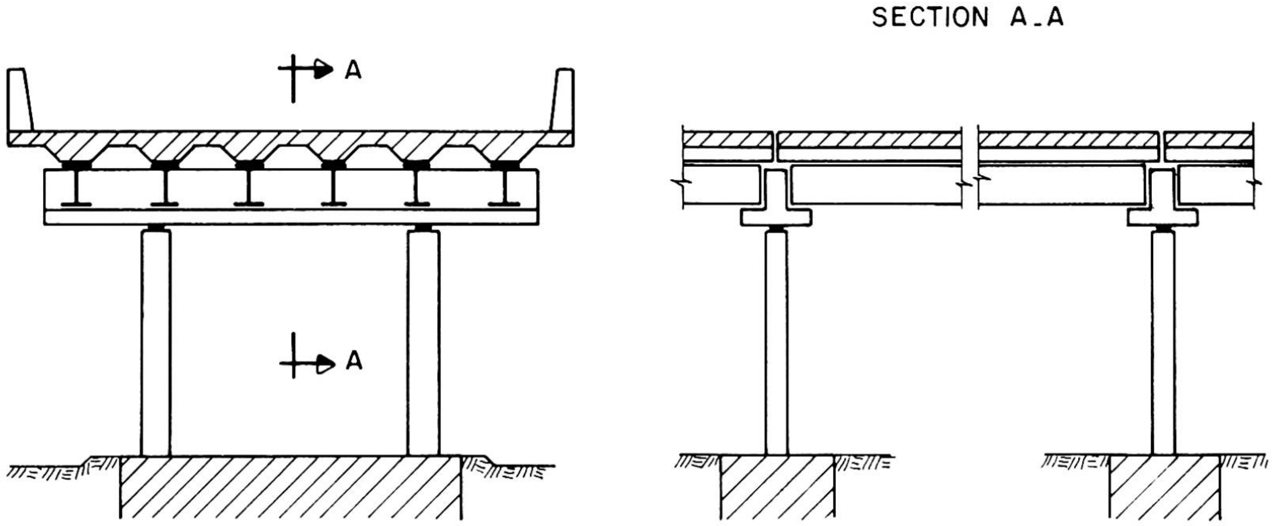
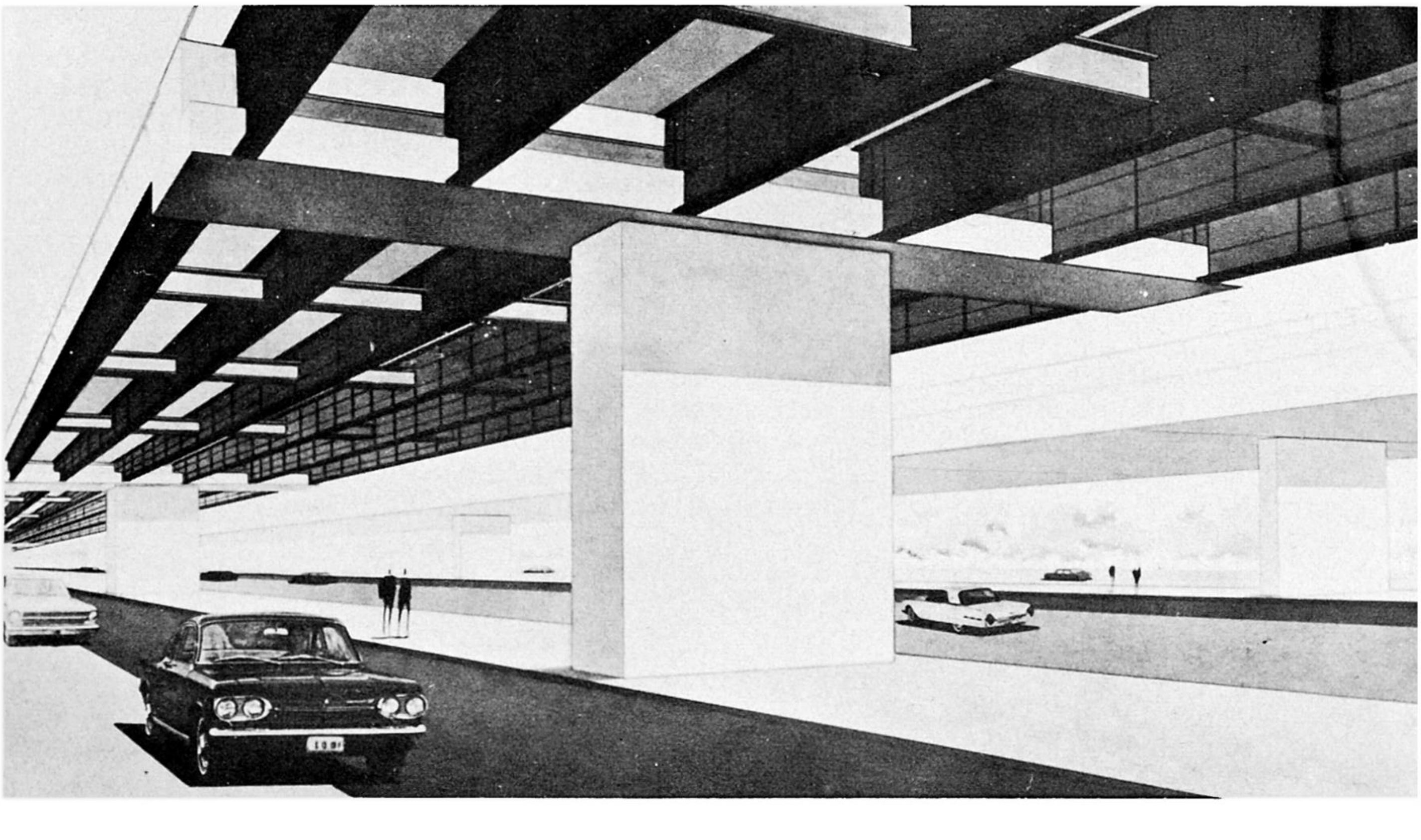
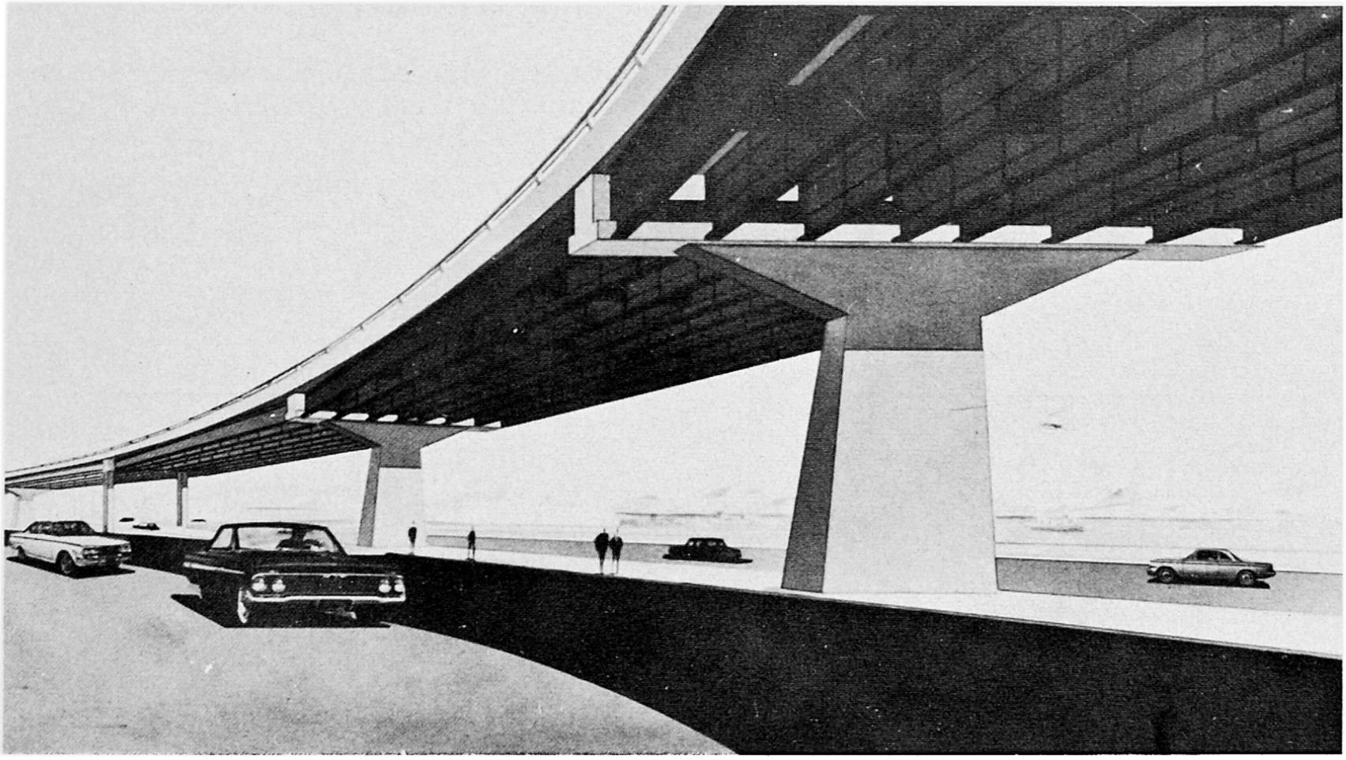


FIG. 1

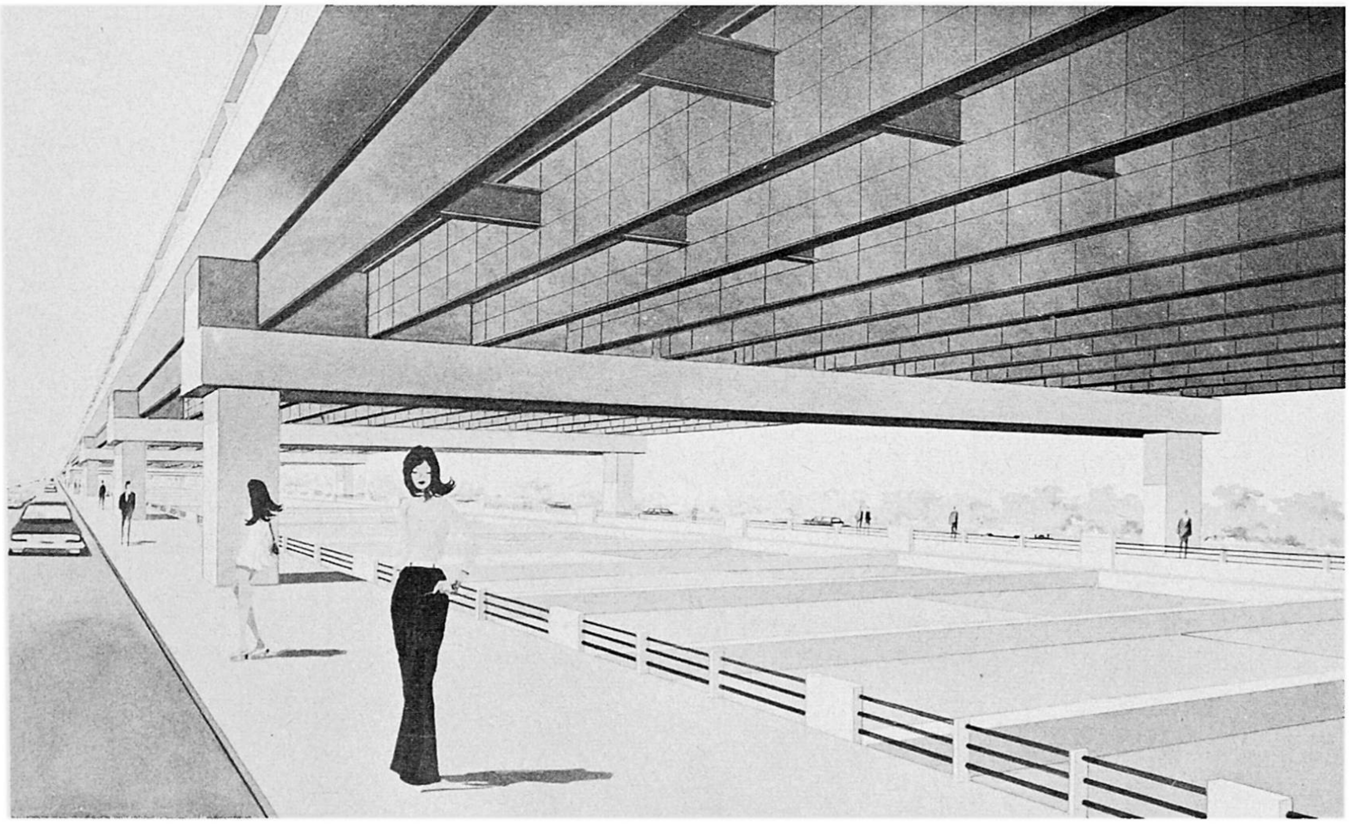
The photographs included illustrate additionally the aspects of the structural solution.





The above photographs refer to solutions with spans close to 40m and a width of the roadway of 20m.

The next photograph refers to a solution with both spans and roadway width of 40m.



It covers a river inside the town, in order to avoid expropriations.

In order to satisfy the requirements stated in the previous item, the superstructure consists of simply supported spans.

The roadway slab, according to the particular situation of the span under consideration, can be cast "in situ" or precast in sections of dimensions compatible with the handling equipment.

In such a case, holes will be left in the slab for positioning the connectors, after the slab is brought to position. These holes can be filled with concrete or epoxy (Fig. 2,a).

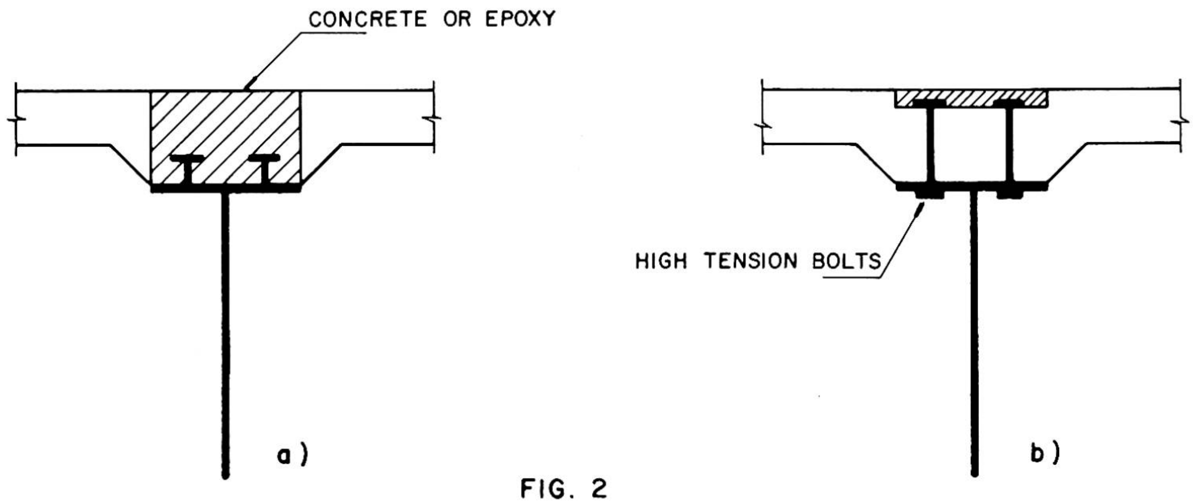


FIG. 2

In order to allow for a possible removal of the roadway slab at a later stage, the connectors could be replaced by high tension bolts, as suggested in Fig. 2,b.

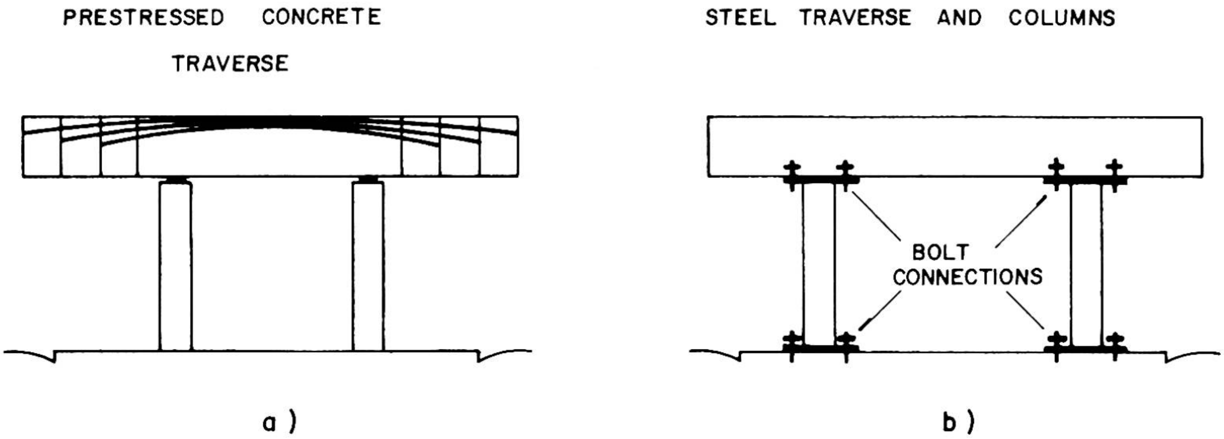


FIG. 3

Figure 3 illustrates the structural solution for the traverses. In case "a", the traverses are prestressed by segments. In case "b", both columns and traverses are in steel, with bolt connections, in order to make removal possible.

In order to reduce the weight of steel in the main beams, a movable truss can be used in order to support the steel beams,during casting of the roadway slab (Fig. 4).

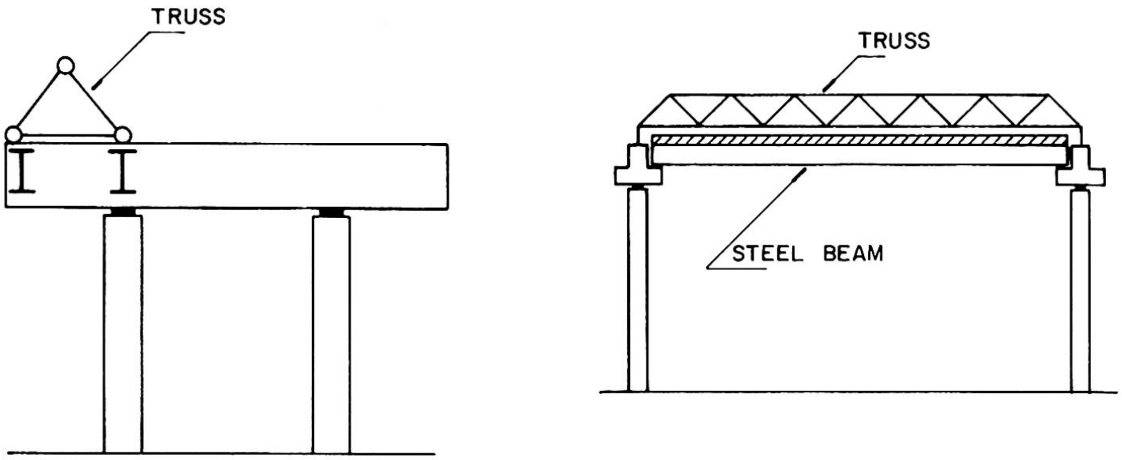


FIG. 4

In this case, the dead load would also be carried by the composite structure.

3 - <u>CONCLUDING REMARKS</u>

The solution discussed was compared economically with several other alternatives and proved to be very advantageous.

A particular advantage of the system, besides the previous ones already mentioned, is that large construction sites are avoided, an important fact for urban areas.

The steel beams and the precast elements are taken to the site only at the time of erection.

SUMMARY

A flexible system of composite construction for urban expressways is discussed. The main features of the structural solution are presented, as well as some remarks on the construction method.

RESUME

Un système de constructions mixtes est présenté pour la construction de routes urbaines surélevées. Les caractéristiques et les procédés de construction sont présentés brièvement.

ZUSAMMENFASSUNG

Eine flexible Verbundlösung für Hochstrassen in städtischen Bereichen wird erörtert. Die massgebenden Eigenschaften der Lösung, sowie das Konstruktionsverfahren, werden kurz dargelegt.

La standardisation modulaire évolutive. Application dans le domaine des ouvrages d'art

Entwicklungsfähige Modular-Standardisation. Anwendung im Brückenbau

Evolutive Module Standardization Applied to Bridges

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A. Considérations générales sur la standardisation

1) Utilité de la standardisation

a) Réduction du coût des études

En plus de la réduction de coût évidente due au fait qu'une seule étude sert pour un grand nombre d'ouvrages ou de parties d'ouvrages, la standardisation permet de diminuer sensiblement le volume des études annexes telles que les études de soumission, d'exécution, de vérification, d'équipements etc...

b) Amélioration de la qualité des études.

Etant donné que les frais d'études s'amortissent sur un grand nombre d'ouvrages, il est possible et rentable de consentir pour ces études, une dépense sensiblement plus importante, et par conséquent, de les pousser beaucoup plus loin au profit de la qualité, de la sécurité et de l'économie.

c) Réduction des délais d'étude.

Quand on réutilise un projet tout étudié, ou dont une partie est déjà étudiée, il est possible de réduire, voire de supprimer Le délai habituellement nécessaire pour les études, et de commencer directement à construire.

d) Réduction du coût de l'exécution.

La possibilité de construire en série permet une réutilisation optimum du matériel, et une augmentation sensible du rendement du personnel.

e) Amélioration de la qualité de l'exécution.

La construction en grande série justifie l'utilisation d'un matériel de première qualité, puisqu'elle en garantit l'amortissement. Ce matériel permet d'obtenir une fabrication de haute qualité. L'expérience acquise après la réalisation d'un certain nombre d'exemplaires d'une construction standardisée permet également une amélioration de l'exécution.

f) Correspondance entre éléments et remplacement de ceux-ci facilités.

A condition de tenir compte des problèmes de liaison lors de l'étude des éléments standardisés, la standardisation peut favoriser la correspondance entre les diverses parties des constructions, ainsi que le remplacement de ces éléments. (par exemple en cas de défectuosité).

g) Possibilité de stockage.

Le nombre d'éléments standards différents étant par définition fort réduit, il devient possible de les fabriquer à l'avance et de les stocker, de manière à toujours disposer des éléments dont on a besoin. Il devient également possible d'étaler la fabrication dans le temps, donc d'éviter le suréquipement et l'irrégularité d'emploi de la main-d'oeuvre.

2) Inconvénients de la standardisation

a) Manque de souplesse d'adaptation aux conditions particulières.

Un ouvrage standard sera rarement aussi bien "ajusté" au site qu'un ouvrage sur mesure.

b) Surabondance de dimensions et de résistance.

On ne dispose pas toujours d'une solution standard correspondant exactement aux conditions minimales imposées, aussi y a-t-il forcément un certain gaspillage de matière.

c) Danger de stagnation technique.

Quand on dispose de projets tout étudiés, la tentation est forte de continuer à les utiliser, même s'ils présentent des défauts par rapport à des projets meilleurs que l'expérience acquise et les progrès de la technique permettraient de réaliser.

d) Monotonie d'aspect.

L'esthétique des ouvrages est plus satisfaisante quand ils sont en harmonie avec le site et bien adaptés à leur fonction et aux conditions particulières locales.

3) Difficultés de la standardisation.

Le grand obstacle à la standardisation est la diversité de ce que nous appelons les "données" c'est-à-dire des conditions imposées par la disposition des lieux, la nature du sol, les caractéristiques des voies portées et franchies, les caractéristiques du trafic etc... Les paramètres qui caractérisent un ouvrage sont très nombreux et le nombre de valeurs qu'ils peuvent prendre est illimité.

B. Solutions de principe pour résoudre les difficultés et réduire les inconvénients.

1) Limitation du nombre de cas différents.

Pour réduire le nombre de cas donc le nombre de combinaisons, il faut réduire à tout prix le nombre d'éléments à un strict minimum.

Il faut réduire le nombre de variables et le nombre de valeurs que peuvent prendre ces variables. Ceci est obtenu très efficacement par la modulation.

2) <u>Choix de solutions polyvalentes.</u>

Quoique l'on fasse le nombre de cas même réduit au minimum sera encore relativement grand, il faut donc rechercher des solutions polyvalentes, faire choix d'ensembles composés d'un nombre aussi petit que possible d'éléments différents et présentant une grande souplesse d'adaptation. Les ensembles seront alors différents comme les cas mais leurs éléments seront identiques.

3) <u>Division modulaire des projets et réalisations.</u>

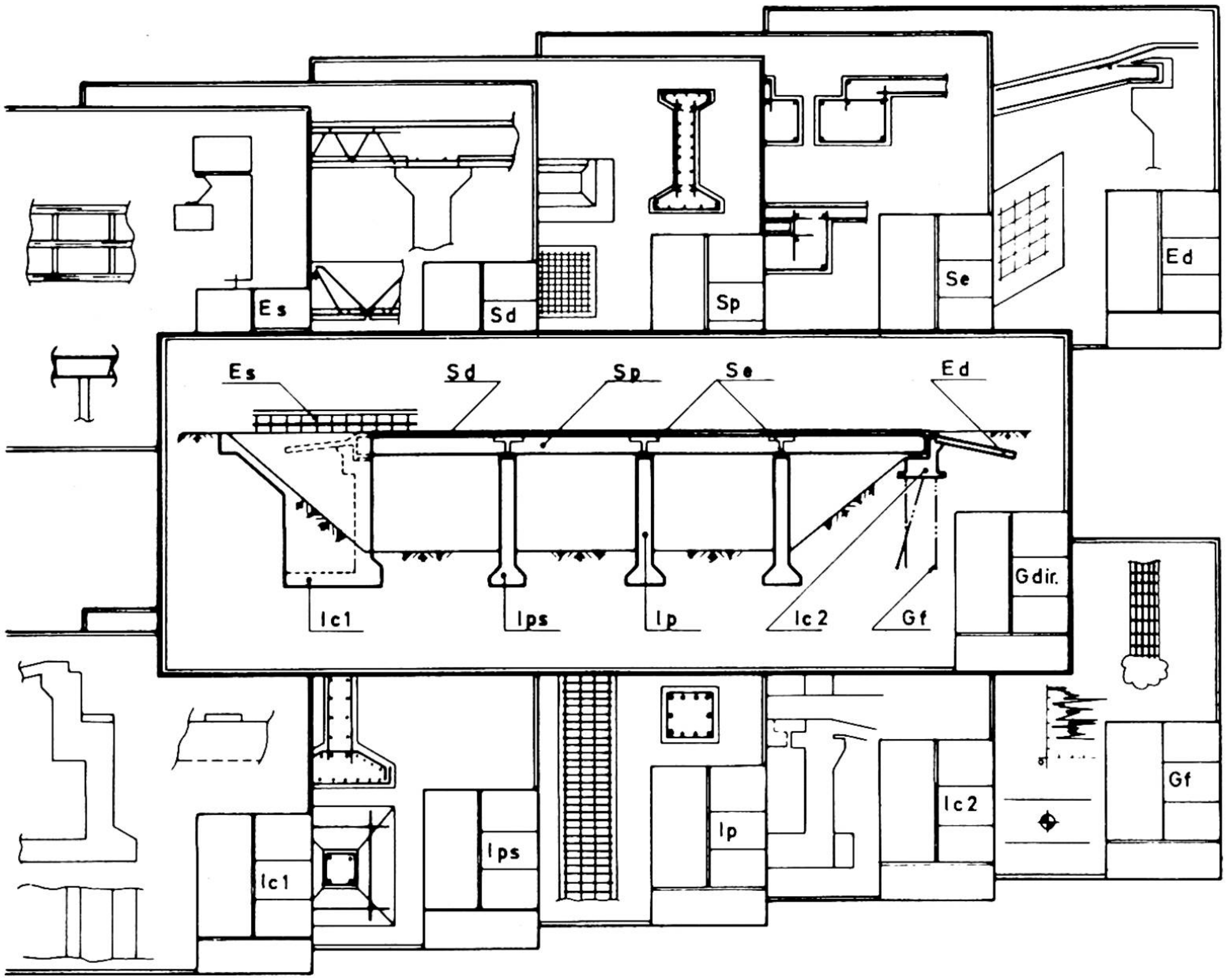
Il faut adopter une division modulaire des projets et des réalisations c'est à dire en respectant des règles de coordination qui permettent aux différentes parties de se raccorder à d'autres.

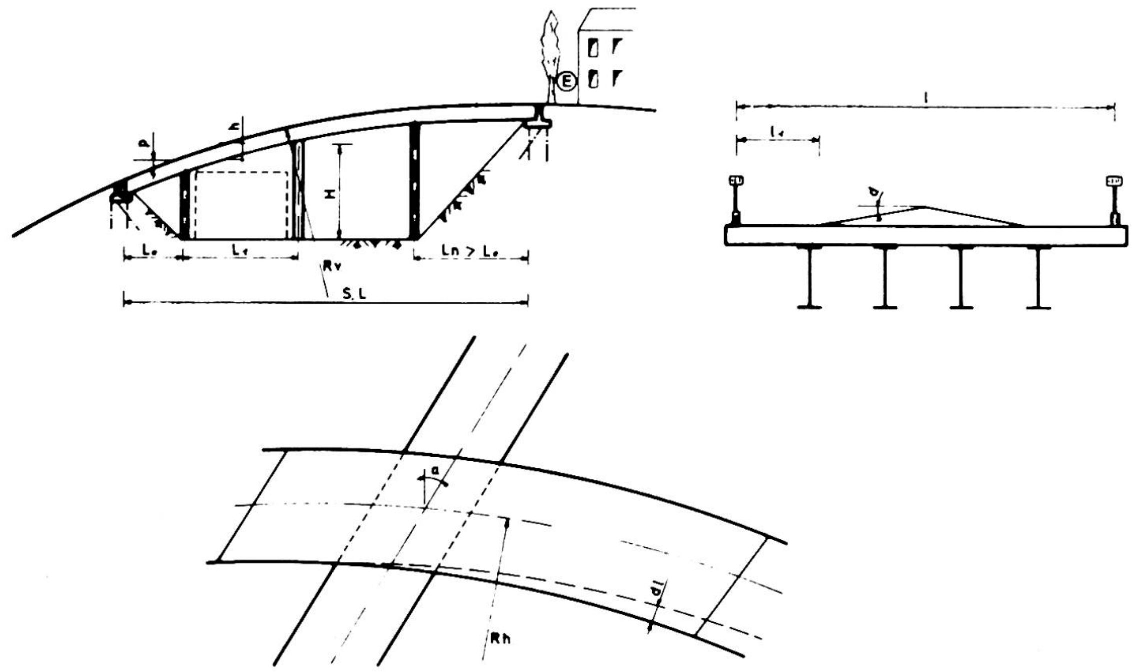
4) <u>Caractère évolutif à donner à chaque partie et au schéma d'assemblage.</u> Pour éviter la stagnation, on doit veiller à ce que chaque partie et le schéma d'assemblage puissent être améliorés indépendamment chaque fois qu'une solution meilleure sera trouvée. En ce qui concerne l'aspect, la souplesse de la formule proposée permet également une diversité suffisante pour éviter une trop grande monotonie.

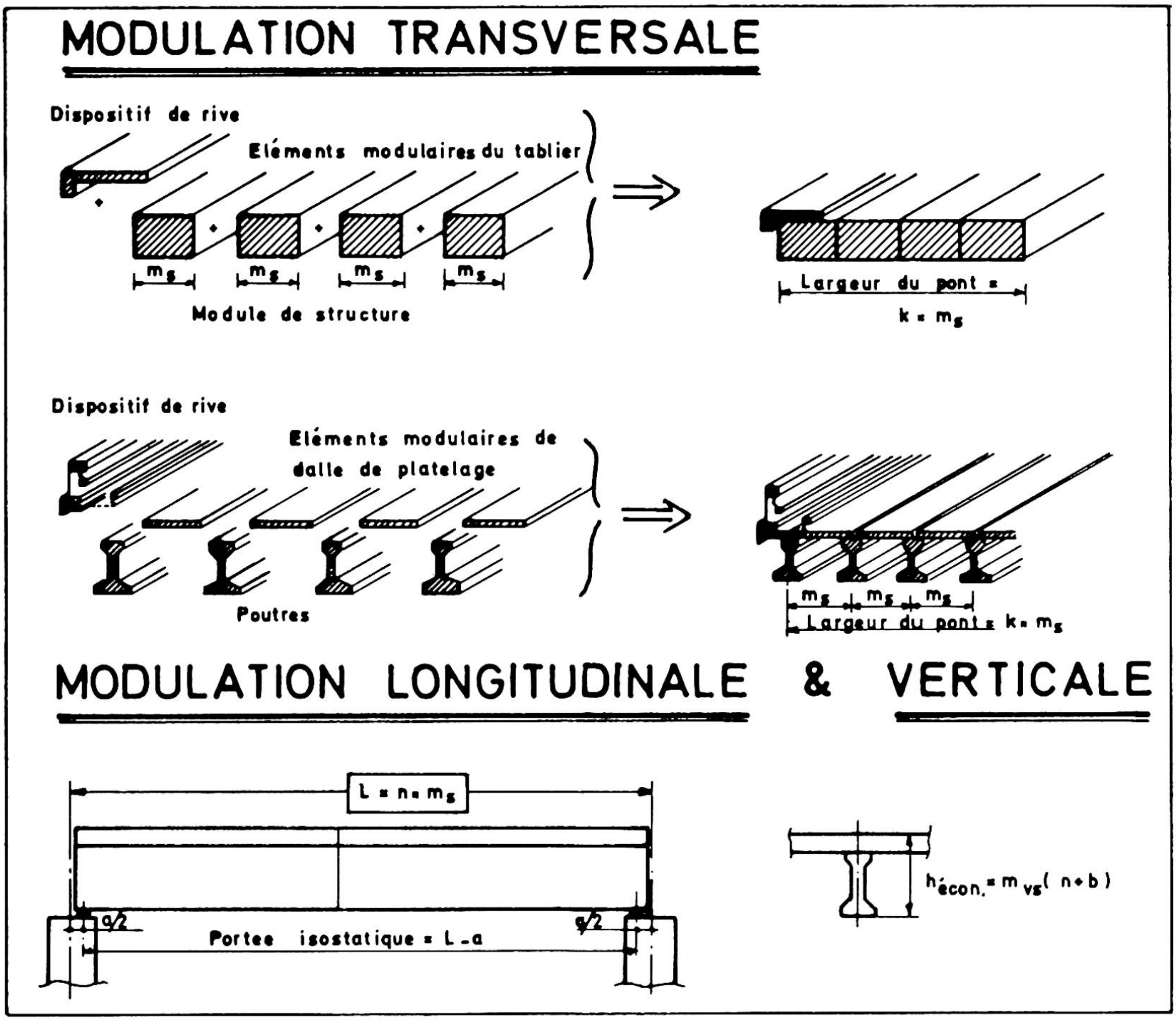
C. <u>Application pratique dans le cas de ponts.</u>

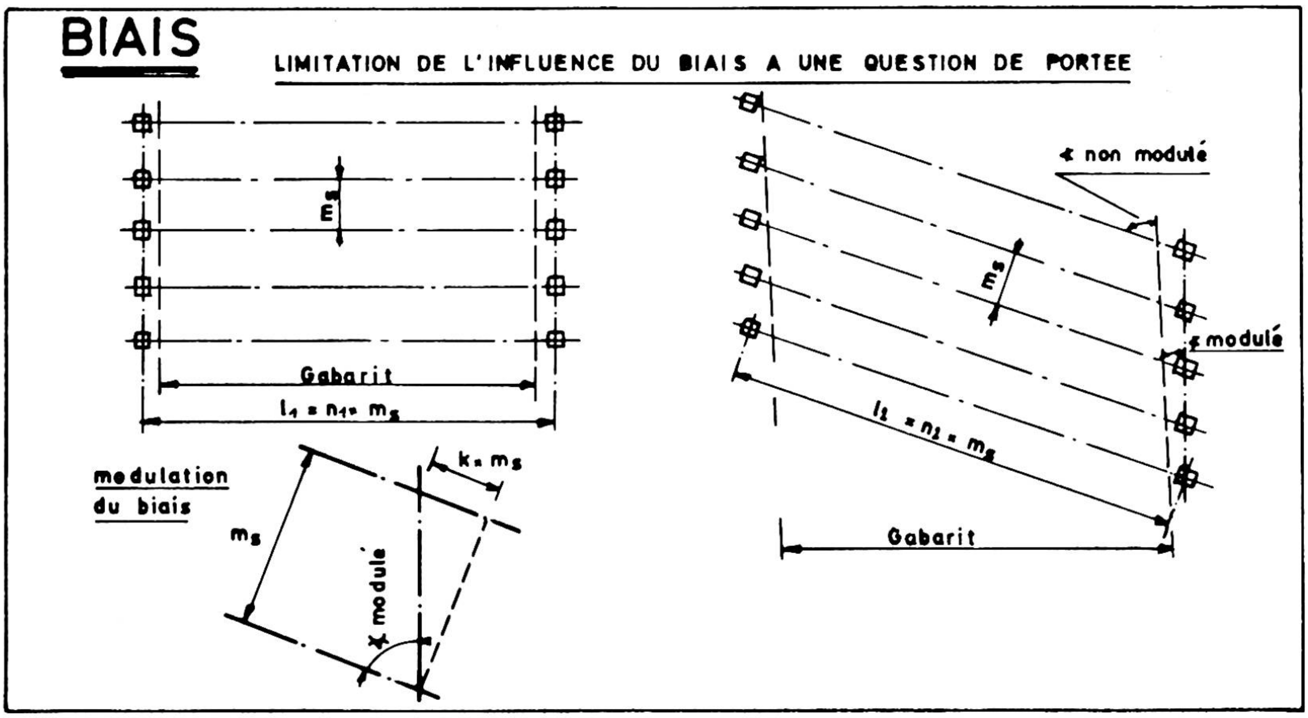
Variables d'un pont donné		Moyen de réduire le nombre de variables et/ou le nombre de valeurs de celles-ci.
L.	portée	modulation (nombre de valeurs réduit)
S.L	longueur	modulation (nombre de valeurs réduit)
l.	largeur	structure indépendante de la largeur grâce à 1.modulation 2.faible entretoisement 3.suppression des chevêtres 4.indépendance des fondations de colonnes 5.groupement des dispositifs de rive en un ensemble standard
h.	haut.tablier	liée à la portée:élancement réduit constant
a.	biais	1.modulation(nombre de valeurs réduit) 2.problème réduit à une question de portéesgrâce à conception adéquate:pas d'entret.nide chevêt. ni de joints sur piles,ni de fondation commune pour les piles.
d.l	larg.variable	dispositifs de rive à largeur variable
Rh	rayon en plan	dispositifs de rive à largeur variable
d	dévers transv.	1.pout.à niv.differ.-2.surch.de profilage.
l1	larg.trottoirs pist.cyclab.	modulation
H	haut.au-dess sol	piles prismatiques et modulation.
P	pent.prof.en long	portées modulées choisies suivant nécessité
Rv	ray.courb. vert.	surcharge de profilage prévue.
E	caractér.d'environnement.	solution de rechange à poutre de hauteur réduite.

DIVISION MODULAIRE D'UN PROJET DE PONT






MODULATION TRANSVERSALE
Dispositif de rive
Eléments modulaires du tablier
m_s
m_s
m_s
m_s
Module de structure
Largeur du pont =
$k \cdot m_s$
Dispositif de rive
Eléments modulaires de dalle de platelage
Poutres
m_s
m_s
m_s
Largeur du pont = $k \cdot m_s$
MODULATION LONGITUDINALE & VERTICALE
$L = n \cdot m_s$
a/2
a/2
Portee isostatique = L - a
$h_{écon.} = m_{vs} (n+b)$


BIAIS
LIMITATION DE L'INFLUENCE DU BIAIS A UNE QUESTION DE PORTEE
E_s
Gabarit
$l_1 = n_1 \cdot m_s$
modulation du biais
m_s
$k \cdot m_s$
∡ modulé
∡ non modulé
m_s
∡ modulé
$l_2 = n_2 \cdot m_s$
Gabarit

Résultats obtenus en Belgique en ce qui concerne les ponts.

Depuis deux ans environ, on a puobtenir que les données routières (largeur des profils en travers, entredistances des supports, hauteurs libres, biais) soient modulées (à 3,60m en général).

Il s'agissait là de la condition sine-qua-non d'une véritable standardisation des ponts.

En ce qui concerne l'étude proprement dite des ouvrages, les autres moyens cités dans le tableau ci-dessus ont été utilisés systématiquement. En première phase, l'effort a surtout porté sur les ponts en poutres préfabriquées pour lesquels la standardisation est plus aisée et plus directement fructueuse. Mais les recherches et les applications sont également poussées pour les autres types d'ouvrages.

D'une manière générale, les résultats sont excellents. Tous les ponts étudiés depuis la modulation des données en profitent pour au moins une partie de leurs éléments (équipements, dalles, poutres entièrement préfabriquées (40% des ponts environ), colonnes, semelles, culées, coffrages, etc.). Certains sont complètement standardisés.

Quand nous parlons de ponts ou d'éléments de ponts entièrement standardisés, il s'agit réellement de constructions et d'éléments qui sont reproduits identiquement. Le stade des poutres qui utilisent les mêmes coffrages ou simplement le même profil est depuis longtemps dépassé.

RESUME

Une véritable standardisation peut être fort utile mais est souvent très difficile par suite de l'infinie diversité des données, et peut présenter aussi des inconvénients. Des solutions ont été trouvées pour résoudre ces difficultés et réduire les inconvénients: réduction du nombre de valeurs de variables, structures souples constituées d'un nombre minimum d'éléments différents, division modulaire, caractère évolutif.

ZUSAMMENFASSUNG

Eine vollkommene Standardisierung kann sehr nützlich sein, ist aber infolge der starken Verschiedenheit von Ausgangswerten oft sehr schwierig und kann sogar Nachteile mit sich bringen. Es wurden Lösungen gefunden, um diese Schwierigkeiten zu bewältigen und die Nachteile zu vermindern: Verkleinerung der Anzahl von Parametern, anpassungsfähige Struktur, bestehend aus einer minimalen Anzahl verschiedener Teile, modulare Unterteilung, entwicklungsfähiger Charakter des Systems.

SUMMARY

A real standardization can be very useful but is often very difficult because of the great diversity of the data given, and can also present some disadvantages. Some solutions have been found to solve these difficulties and to reduce the disadvantages: reduction of the number of dimensions of the variables, adaptable structures consisting of a minimum number of different elements, module division, and evolutive character of the system.

Design and Construction of the Hokawazu Bridge

Conception et réalisation du pont de Hokawazu

Entwurf und Ausführung der Hokawazu-Brücke

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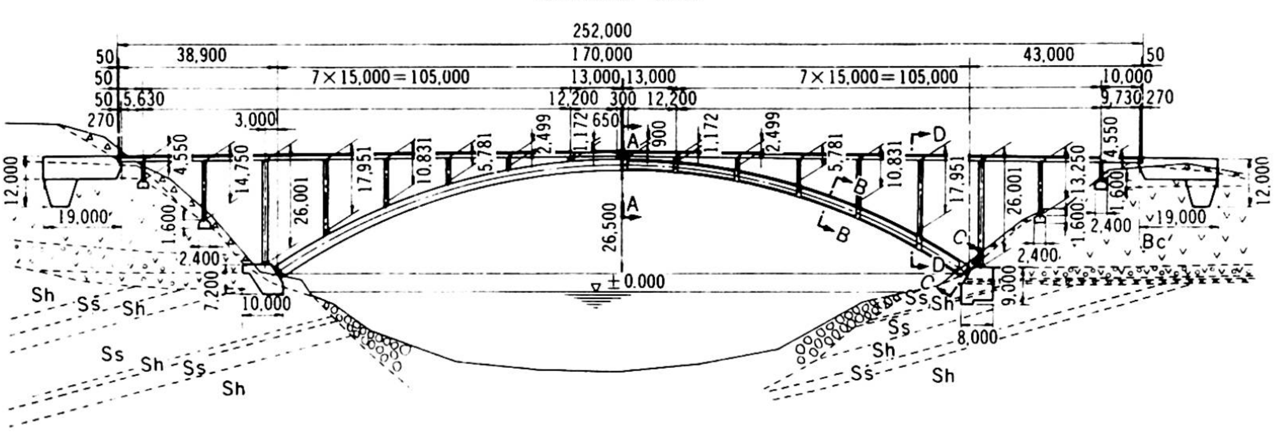
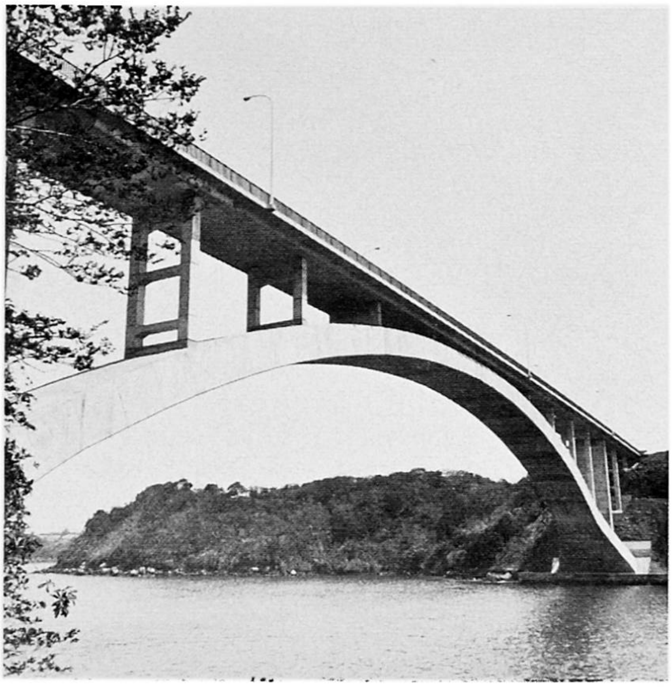
1. Preface

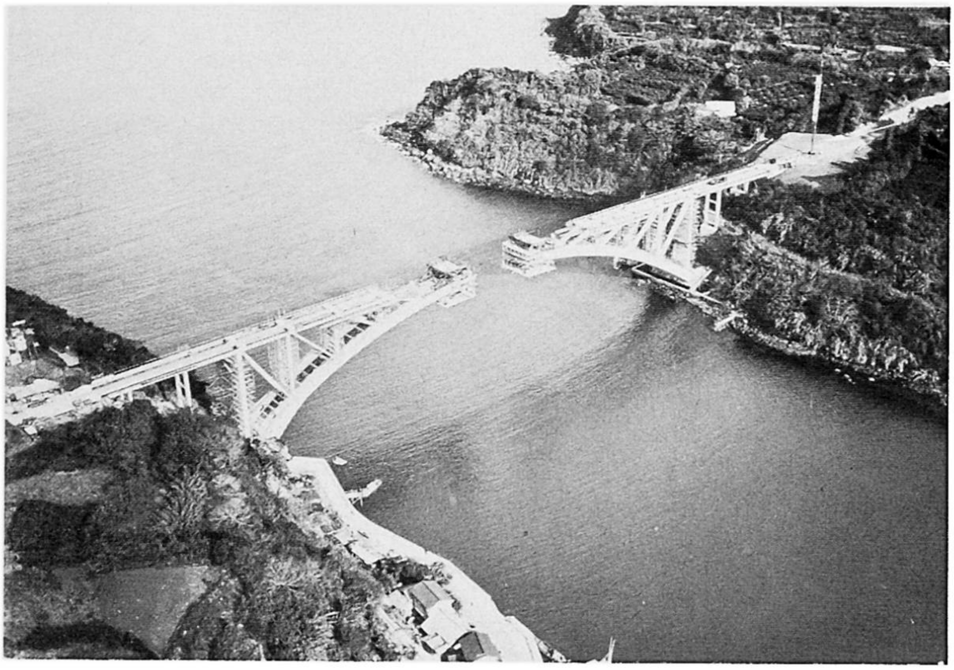
The National Highway Route 204 starts from Karatsu City in Saga Prefecture, goes along the Sea of Genkai, runs through Imari City in the same prefecture, and leads to Sasebo City in Nagasaki Prefecture, and the total length of the route reaches upto 154 km. The Hokawazu Creek made the route discontinuous between Chinzei and Genkai at Higashi-Matsuura County in Saga Prefecture, and transportation between these two towns depended upon ferryboats only. But the openning of the Hokawazu Bridge on May 1, 1974 dissolved the inconvenience in transportation in this area and the bridge is expected to be a "bridge" toward the prosperity of the region.

The Hokawazu Bridge, with a center span of 170 m and a total length of 252 m, is the longest reinforced concrete arch bridge in Japan, and was built by means of a new construction method unprecedented in the world. Since the adoption of the traditional staging construction method was difficult for the reason that the bridge is located on the sea and its floor level is too high (about 50 m above the seabed) to build a staging, a cantilever construction method was adopted in which the constructing segments formed of an arch rib, struts and floor slabs were supported by prestressing steel bars and the overhanged bodies extended their length step by step from both the shores toward the center until the final segment was placed at the center.

GENERAL VIEW







(1) Intent of the design

Reinforced concrete arch bridges are generally constructed with the help of stagings such as "arch center". But in this bridge, a cantilever construction method was adopted, in which the constructing segments formed of an arch, struts and floor slabs were supported by prestressing bars and, therefore, it was necessary to analyze each of the struts and the floor slabs as well as the arch rib, as a part of a cantilevered structure.

The design calculation by a computer with a large capacity was made for every temporary structural system corresponding to the growth of the arch rib, struts and slabs, conditions of falsehood and other equipment, as well as for the final structural system.

(2) Outline of the design

1) Arch rib

The arch rib, with a cross section of two boxes, was designed as two hinged structure.
The span length and the rise of the arch are 170 m and 26.5 m, respectively. The arch rib has a height of 3.0 m at its springings, 2.4 m at the crown, and has a width of 8.0 m except the portions of the nearest 19 m from the springings where it was increased lineally from 8.0 m to 16.0 m providing the maximum at the springings for the reason of improvement of stability against lateral seismic and wind forces perpendicular to the bridge axis.

2) Struts and piers

Struts on the arch rib range from 17.951 m to 0.635 m in their heights, and were designed as rigid frame structures of one to five stories. Top and bottom ends of each strut have steel pin bearings so that the strut can behave as a rocking pier absorbing the large amount of bending moments due to the inclination of the strut by the over-raising of the structure during the construction and by the influence of temperature changes, creep and shrinkage of the concrete.

3) Floor Slab

Continuous hollow slab, over nine spans and 60 cm thick, were adopted because it was important to decrease the dead weight of the slab in such cantilever construction.

4) Abutments

Abutments of large dimensions at both ends of the bridge were necessary as anchorages against the overturning of the cantilevered structural parts during the construction, and such large dimensions are not necessary for the stability of the bridge after the completion.

5) Abutments for arch rib

The maximum axial compressive force in the arch rib at its springings is approximately 5000t at the time of construction and 6700t at the final stage of completion. The axial force is transmitted to the abutment for the arch rib through a large bearing which consists of four steel pin bearings.

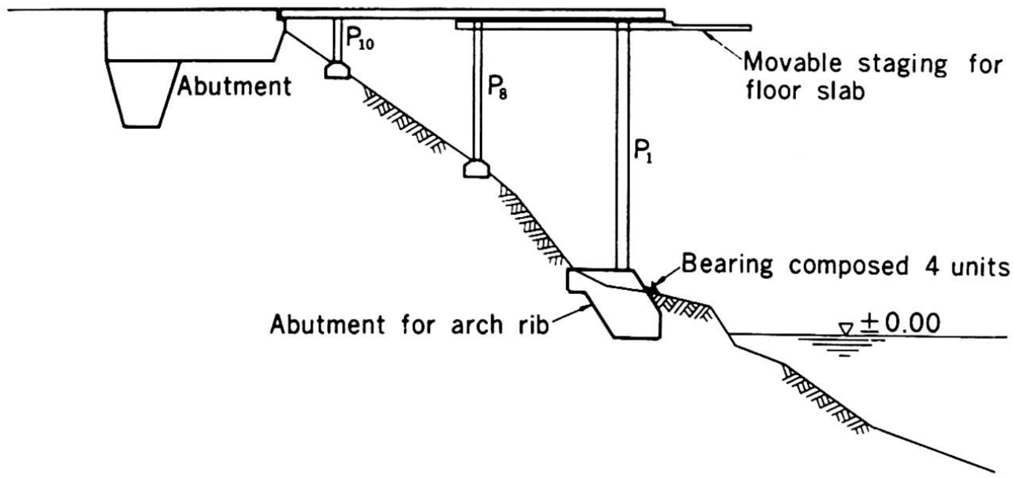
The dimensions of the abutments reach 20 m in width, 9 m in height and 8 m in length.

Construction

(1) Construction process

1) Construction of side spans

i. Concrete placing for abutments and piers
ii. Placement of 76 prestressing steel bars on the surface of the floor slabs.
iii. Placement of four bearings (capable of 1655t reaction by a bearing) for the arch rib.



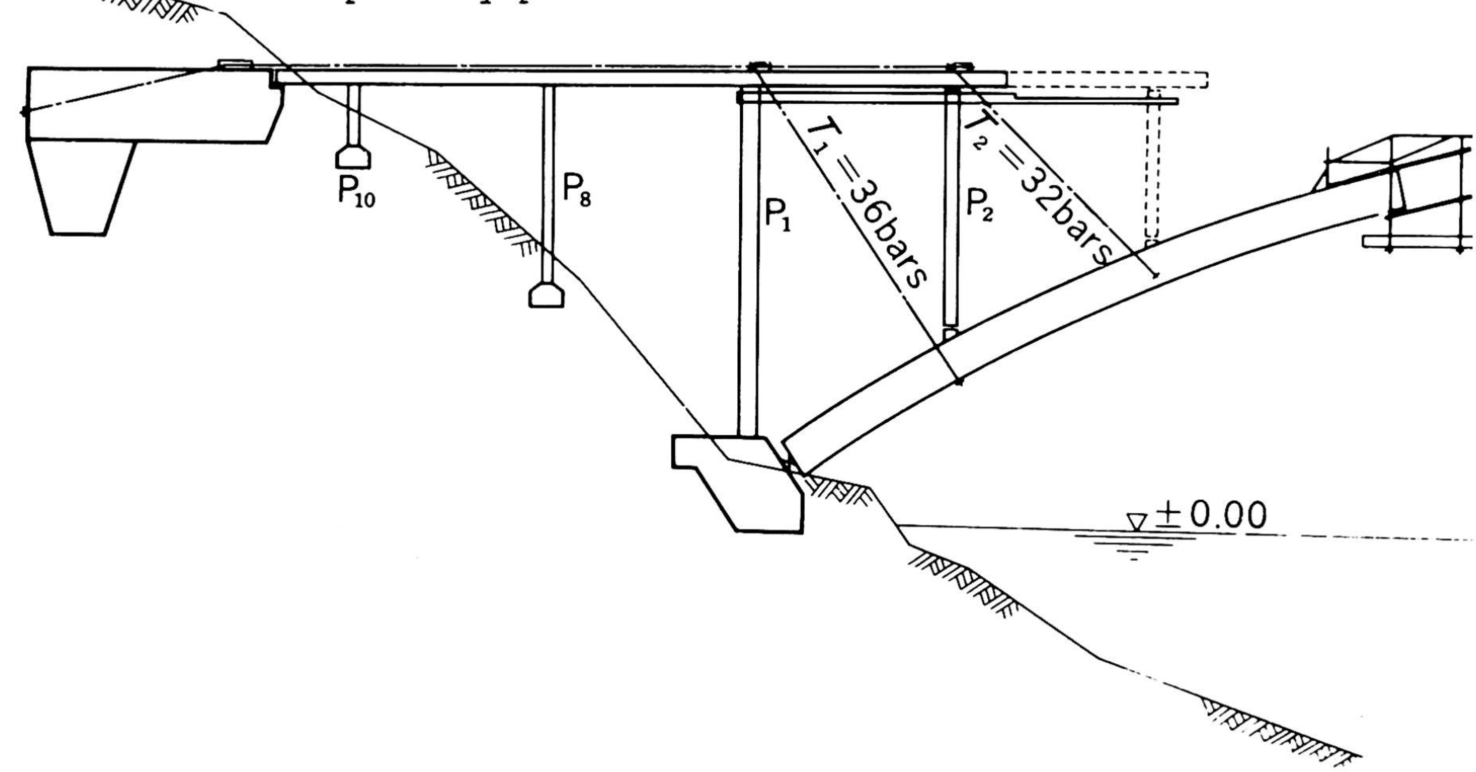
2) Construction of a part of the arch rib on suspended falseworks

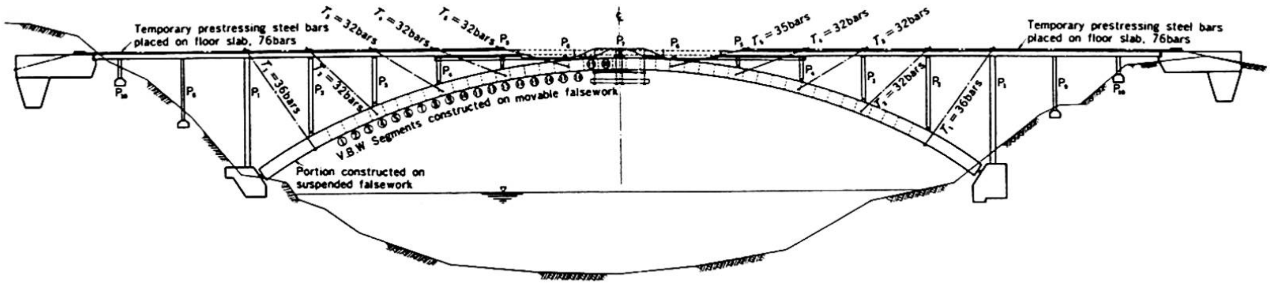
   i. Placement of steel panel girder falseworks suspended by 36 inclined prestressing steel bars, T_1, in the portions between P_1 and P_2.

3) Construction of the consecutive part of the arch rib on movable falseworks

   i. Construction of the arch rib between P_1 and P_2 by cantilever construction method on the "Wagen".
   ii. Construction of struts, P_2, followed after the above stage.
   iii. Construction of the floor slab between P_1 and P_2 on movable falseworks.
   iv. Placement of 32 inclined prestressing steel bars, T_2, and of prestressing steel bars on the floor slab between T_1 and T_2.
   v. Advancing the above processes and completing the arch rib except the arch crown.

4) Construction of the arch crown and re-movement of equipment and temporarily placed steel bars.





POSTSCRIPT

There have been a lot of reinforced concrete arch bridges since considerably old times. In every case of them the major problem to be solved was how to erect the structure. But we are sure that the erection method developed in this Hokawazu Bridge work can be a step to solue it. As arch-type bridge is a most hopeful structural system among concrete bridges of large span which are supposed to become very popular in future, we would be much pleased if the data of this bridge could greatly contribute the establishment of an effective erection method.

REFERENCES

Y. Inoue and Y. Miyazaki: "Construction of the Hokawazu Bridge", Roads, February 1974

Y. Miyazaki and T. Igarashi: "Design and Construction of the Hokawazu Bridge, (I) and (II)", Bridges and Foundations, Vol. 8, No. 7 & No. 8, 1974

Mario d'Aragona: "La construzione dell' arco del ponte Van Staden's in Sud Africa", L'Industria Italiana del Cemento, Anno XLl, dicembre 1971

SUMMARY

This paper describes a special method applied to the construction of a reinforce concrete arched bridge with a length of 252 m and a central span of 170 m. The conventional staging method is not employed in the bridge erection and the whole bridge is overhung from the two banks and connected at the central crown part.

RESUME

Cet article décrit un procédé spécial utilisé pour la construction d'un pont en arc en béton armé, de 252 m de long et d'une travée centrale de 170 m. On n'utilise pas la méthode classique de l'échafaudage dans la construction de ce pont. Le pont entier est construit en encorbellement à partir des deux rives.

ZUSAMMENFASSUNG

Dieser Bericht beschreibt ein Spezialverfahren, das beim Bau einer Bogenbrücke aus Eisenbeton mit einer Länge von 252 m und einer zentralen Spannweite von 170 m angewendet wurde. Die herkömmliche Gerüstaufstellung wird hier vermieden. Die ganze Brücke wird von den zwei Flussufern her vorgebaut und im Bogenscheitel verbunden.

Mechanization of Bridge Construction by Use of Large Prefabricated Blocks

Mécanisation de la construction de pont par utilisation de gros éléments préfabriqués

Mechanisierung der Brückenmontage unter Anwendung grosser Fertigteile

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1. Bridge Construction in Large Prefabricated Blocks in Japan

A conspicuous trend characterizing bridge construction in Japan lately is the use of increasingly larger prefabricated blocks owing to mechanization of erection work. (See Table 1.)

Naturally, the construction of bridges in large blocks is nothing new. However, a distinct conceptual difference lies between the past and the present. In the past, the method of constructing bridges by the use of large blocks had been either an inevitable alternative where other construction methods appeared unfeasible, or had been adopted for very limited applications indeed.

Table 1. Examples of steel bridges constructed by the large block erection method in Japan (1974-75)

Name of bridge	Year of construction	Type	LENGTH OVERALL	Main particulars of erection block	Outline of erection work
Osaka Port Bridge	1974	Cantilever truss bridge	m 980 (235+510+235)	L x B x D— 192.1 m x 23.9 x 26.8 m Weight of max. block 4,500 t	Transshipment onto deck barge by a 3,000-ton+2,500-ton floating crane; hoisted by 8 units of 132 kW double-drum winch
Daikoku Bridge	"	Cable-stayed Bridge	m 265.88 (165.38+100.50)	L x B x D—105 m x 24.2 x 2.78 m Weight of max. block 950 t	Erected by a floating crane with a lifting capacity of 1,500 tons
Katagami Bridge	"	4-span continuous steel box girder	m 475 (95+160+120+100)	L x B x D—131.8 m x 9.3 x 7.1 m Weight of max. block 552 t	Erected by a floating crane with a lifting capacity of 1,300 tons
Suehiro Bridge	"	Cable-stayed bridge	m 470 (110+250+110)	L x B x D—52 m x 18.5 x 2.8 m Weight of max. block 340 t	Floating crane method, 1,000 ton lifting capacity
Ohgishima Bridge	"	3-span continuous steel box girder	m 184 (50+84+50)	L x B x D—184 m x 15.8 x 3.392 m Weight of max. block 1,200 t	Floating crane method; 1,500 ton lifting capacity
Arakawa Coast Bridge	1975	Cantilever truss bridge	m 840 (100+120+125 +150+125+120+100)	L x B x D—197.8 m x 48.5 x 21.57 m Weight of max. block 4,250 t	the largest girder erected by three floating cranes-3,000-ton+2 x 1,500 ton
Oshima Bridge	"	3-span continuous truss bridge	m 725 (200+325+200)	L x B x D—212.5 m x 11.0 x 38.3 m Weight of max. block 1,895 t	Floating crane method, 3,000 ton lifting capacity
No. 2 Maya Bridge	"	3-span continuous steel box girder	m 360 (75+210+75)	L x B x D—127.5 m x 18.0 x 7.5 m Weight of max. block 1,700 t	Floating crane method, 3,000 ton lifting capacity
Kamome Bridge	"	Cable-stayed bridge	m 440 (100+240+100)	L x B x D—75.6 m x 20.5 x 3.1 m Weight of max. block 600 t	Floating crane method 1,300 ton lifting capacity
Hirato Bridge	"	Suspension bridge	m 665 (center span 465.4 m)	L x B x D—63.0 m x 17.5 x 4.0 m Weight of main tower 567.3 t	Floating crane method appolied to the main tower only; 1,300 ton lifting capacity
Rokko Bridge	"	Cable-stayed bridge	m 400 (90+220+90)	L x B x D—93.6 m x 24.1 x 9.348 m Weight of max. block 1,450 t	Floating crane method; 3,000 ton lifting capacity
Kajima Bridge	"	3-span continuous truss bridge	m 340 (69.2+170+99)	L x B x D—180 m x 7.2 x 17.5 m Weight of max. block 634 t	Floating crane method; 3,000 ton lifting capacity

By contrast, today the method is rather a preferred choice which the designer makes on his own initiative from among the many methods available.

In short, the method of constructing bridges in large prefabricated blocks has come back into the picture as a kind of popular prefabricated bridge construction method.

True, even in the past, the structural members of a steel bridge were prepared almost entirely at a shop for simple assembly at site. In this sense, the conventional method may be conceived of essentially comprising a prefab construction method. Only, most conventional methods required a larger proportion of field assembling work to be achieved under stringent working conditions, so the construction period was naturally longer.

The bridge construction method using large prefabricated blocks was developed in Japan in the backdrop of the situation described above as a means to capitalize fully on the prefabricated bridge construction method by improving it as close as possible to perfection.

As observed from another angle, the emergence of the Japanese method of constructing bridges in large prefabricated blocks may be construed as having been stimulated by the monumental success achieved by the Japanese shipbuilding industry which adopted the method of building ships in large prefabricated blocks during the postwar years.

The final assemblage work in shipbuilding up to not so long ago used to be achieved by conveying comparatively small, shop-assembled blocks weighing only a few tons onto the building dock where armies of workers where thrown into action to fabricate the ship.

The ever larger vessels came to be built from year to year, the manhours required for assembling increased steadily owing to the inescapable volume of assembling work required on the building docks where working conditions were cruelly restrained; naturally, the ships under construction came to tie down the building docks for longer and longer periods of time. This worked bitterly against shipbuilding efficiency, for the building dock's turnover rate essentially governs the working efficiency of not only the building dock itself but also of the total workshop including steel material stockyard, machining shop, rigging shop, rigging quay and so forth.

To cope with the situation, a large proportion of assembling work on building docks was transferred to the assembling shop by capitalizing on advanced welding techniques as a means to improve shipbuilding productivity, thus heralding in the shipbuilding industry the method of assembling ships in large prefabricated blocks.

Prefabricated blocks conveyed onto building docks have greatly increased in size, currently weighing some 200 – 300 tons each, and working with such massive blocks has inevitably entailed the use of giant hoisting cranes having tremendous capacities.

The successful adoption of large prefabricated blocks for assembling work by the shipbuilding industry has moved bridge constructors to reappraise the advantages accompanying the method to use large prefabricated blocks. But it was only in the early 1960s that the bridge construction method using large prefabricated blocks, practised by Japanese bridge constructors, attracted worldwide attention.

2. Heavy Equipment for Block Handling

Table 1 clearly indicates the overwhelming number of bridges constructed by the aid of floating cranes. Table 2 lists the types of large-size floating cranes now available in Japan.

These heavy equipment were designed not specifically for bridge construction but for use by Japanese coastal industrial complexes in general, for while the country abounds with waters there is a critical scarcity of natural resources. This compels the nation to import raw materials, to convert them into products having high added value, and to export these products. Accordingly, numerous industries proliferate along coastal regions, including ports, harbors and shipyards.

In addition to the large size floating cranes listed in Table 2, they are available in a wide variety of smaller sizes, from which we can freely select the type and size of equipment meeting the specific needs of our bridge construction project.

Table 2. Specifications of large floating cranes (1,000 tons or more in lifting capacity)

Name	Year of construction	Hull dimensions, L x B x D (m)	Hoisting Specifications		
			Rated load (t)	Outreach (m)	Lifting height at rated load(m)
Kiryu	1969	95.0 x 45.0 x 6.67	3,000	29.00	75.00
Musashi	1974	107.0 x 49.0 x 7.70	3,000	41.50	102.00
No. 25 Yoshida	1972	94.0 x 40.0 x 7.80	2,900	28.00	49.00
Shinryu	1971	80.0 x 34.0 x 6.50	1,500	30.00	65.00
Sagami	1972	80.0 x 36.0 x 6.00	1,500	27.88	65.50
Kenryu	1973	80.0 x 30.0 x 5.50	1,300	26.50	60.00
Nagato	1972	80.0 x 36.0 x 6.00	1,300	32.88	80.15
No. 23 Yoshida	1967	74.0 x 31.0 x 6.00	1,200	21.00	49.00
Shokaku	1964	69.0 x 27.0 x 5.80	1,000	19.00	45.00
No. 80 Hoei	1972	72.0 x 30.0 x 5.30	1,000	24.50	42.00
Nisshin	1972	80.0 x 30.0 x 5.50	1,000	26.50	60.00

Shown in Table 3 are the kinds of operations under taken by floating cranes.

Table 3. Working percentage of large floating cranes by kind of work

Kind of work	Frequency of use	Earning ratio
Civil work at ports and harbours	18 per cent	34 per cent
Cargo handling at ports and harbours	44	20
Shipbuilding	21	17
Installation of structure other than bridges	15	17
Bridges	2	12
Total	100 per cent	100 per cent

Note: Based on investigations of 177 cases (1975)

3. Advantages of Construction in Large Prefab Blocks

The sharp rise lately in labor costs has naturally made itself felt at construction sites. The situation has been aggravated by the tendency of young people to prefer working in tertiary or service industries, with the result that the labor force tends to shift towards a higher age bracket and skilled workers are ever harder to mobilize.

Unlike some other countries, Japan does not employ foreign labor, so the need for labor-saving becomes all the more a crucial matter for every business aiming at rationalization of operations.

While conditions may differ from case to case, Fig. 1 demonstrates typical labor-saving effects of the said method of constructing in large prefabricated blocks over conventional methods. True, the said construction method, while permitting rationalization of construction work on one hand, will inevitably lead to higher costs in pre-erection work, in that a large space of the assembling yard in the shop will be occupied for a long period of time at the sacrifice of other assembling work.

The foundation work will also prove costly because of the necessity to bear up against tremendous weight. In addition, transportation of large blocks by water may sometimes prove more costly than handling small blocks, depending on the conditions involved. This would mean that the economics of adopting the construction method using large prefabricated blocks should be discussed by taking into consideration all the factors involved including manufacturing and transportation.

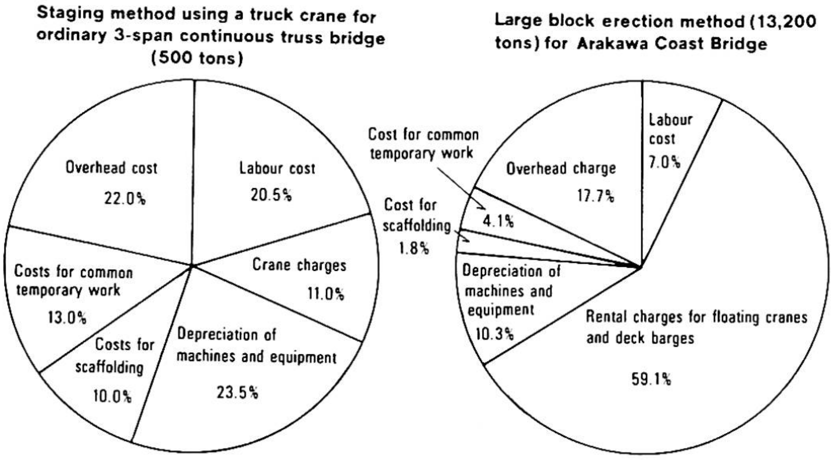

Fig. 1 Cost comparison between large block erection method and conventional erection method

The said large-block method is advantageous in that it shortens the construction time of not only the bridge body itself but also of incidental work to be done after installing the bridge body. Scaffolds and small equipment necessary for advancing remaining construction works such as concrete floor slab laying, for example, can be mounted on the main bridge body at the assembling yard beforehand.

Where the floating crane is concerned, its use in bridge erection will naturally be limited to applications involving waterways. Even in this case, its use may be obstructed by existing bridges. While dismantling its jib for passage under the bridge and subsequent reassembling may be conceivable, the time and costs would in most cases prove prohibitive. In addition, during erection work, the vessel will be completely tied down at the erection site, a cause for additional costs.

In order to cope with these problems, the long practised method of mounting the bridge body on a barge instead of on a floating crane and erecting the bridge body directly on the piers, is adopted. For this, the tidal level difference is utilized and hydraulic jacks are used.

Large floating cranes have deep draught, and as the trim will be changed considerably according to load conditions, they can not be used in shallow waters. Dredging will prove costly in itself, and may conflict with fishing and other local industries.

The lift-up barge shown in Fig. 2, which was used to erect the Keihin Bridge, can be used both for transportation and erection, and can work even in shallow waters as it has an even keel. Since it is not so tall as a floating crane, it can clear bridges having fair clearance.

Fig. 2 Erection of a block of 605 ton by lift-up barge

The lift-up barge transports the bridge block to the construction site while maintaining it at a low position, and lifts the bridge block to the specified position without the aid of any outside equipment.

In Japan, large bridges are mostly built along the coasts or between adjacent islands, and bridge construction projects involving the use of large prefabricated blocks are expected to increase steadily in the years ahead.

Actually, the gross tonnage of steel bridges constructed by the aid of large floating cranes assumes only a small fraction of the total orders for steel bridges, suggesting that bridges which can be constructed by applying the large prefabricated block method are largely limited in terms of structural scale and or geographical conditions.

However, recent bridge construction methods are essentially characterized rather by a wide scope of mechanization and resultant labor saving, a most typical example of which is the construction of bridges by use of large prefabricated blocks with the aid of large floating cranes.

4. Safety Control

The importance of safety control can hardly be over-emphasized. Today, with social responsibility for safety attracting public interest as never before, safety has come to be placed foremost above anything else.

We can never rest assured that the large prefabricated block erection method is intrinsically safer than other methods merely on the fact that no accident occurred in the past. Once it occurs, an accident will prove disastrous not only in terms of direct losses in life and material; the losses sustained by society will be far larger than those invited by conventional construction methods.

No failure being permissible, the large prefabricated block erection method is naturally undertaken under the strictest safety control setup. Namely, not only bridge construction experts but also engineers related to marine vessels, oceanography, transportation and many other fields rally their efforts to establish a foolproof safety control system with their advanced technology and scrupulous care.

Whereas construction by the ordinary method is carried out discretely in veiw of time and space, work by the large prefabricated block erection method is advanced in concentrated time and space, which enables safety control to be achieved more thoroughly.

A sharp reduction in the number of workers through labor-saving measures conduces to reducing the possibilities of human hazards. Also, since mechanization reduces the amount of work done at high places, the degree of human safety is even more amplified.

The fact that construction is concentrated in a short span of time, or as the time required for erection under hazardous conditions will be quite limited, the relative safety of the said construction method against unpredictable natural elements such as wind, wave and earthquake can be increased, with the result that a wider freedom of choice will be in store for meteorological as well as sea conditions.

5. Examples of Bridge Erection Work

Figs 3 the erection work of the Arakawa Coast Bridge. (Constructed jointly by IHI, Mitsubishi Heavy Industries and Yokogawa Bridge Works on order by the Metropolitan Expressway Public Corporation.)

The steel structure of Arakawa Coast Bridge, weighing 13,200 tons, was erected as divided into seven (7) blocks. These blocks were assembled at three coastal assembling yards in Tokyo Bay, then loaded on a large deck barge (DW = 12,000 t) by floating cranes for transportation to the erection site by sea.

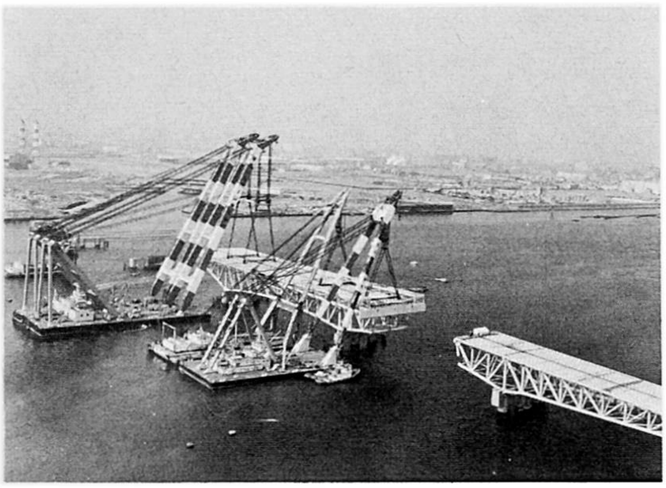

Fig. 3 Erection of the largest block (4,250 ton)

It was on February 11, 1975 that the first block was swung out of the shop onto a deck barge, and the seventh block was erected in the final position on April 23, the same year.

Although the erection work was partly interrupted during that period owing to strong winds, the entire project was executed strictly as scheduled.

When hoisting with three floating cranes the bridge's largest block of 4,250 tons, the load distribution was controlled by means of load meters. By action of the self-regulatory effect based on changes in the draughts, the hoisting loads of the respective floating cranes were successfully contained within their prescribed tolerance ranges as initially planned without any difficulties.

Prospects are now bright for the economical and safe erection of large prefabricated blocks, which may be even larger than those used in the construction of the Arakawa Coast Bridge, through combined use of several floating cranes.

SUMMARY

Examples of bridges constructed by the method of using large prefabricated blocks are given, together with the reasons which have made this method popular lately in Japan. Also given is a list of floating cranes which serve as a key to this method, and their functions. In addition, the large prefabricated block erection method is discussed from the viewpoint of working safety. Finally, the erection of the Arakawa Coast Bridge is introduced as a typical example.

RESUME

Des examples de ponts construits avec la méthode de montage de grands éléments sont donnés, ainsi que la raison de sa large diffusion au Japon. On donne une liste de grues flottantes, comme équipement principal pour cette méthode et de leur fonctionnement. Les problèmes de sécurité sont évoqués en cas d'utilisation de la méthode en question. La construction du pont de Arakawa est présentée comme exemple.

ZUSAMMENFASSUNG

Es wird über Brücken berichtet, die in Grossfertigteil-Bauweise ausgeführt wurden, und zugleich der Grund erörtert, weshalb diese Bauweise zur Zeit in Japan so oft zur Anwendung gelangt. Es wird auch eine Uebersicht über die verfügbaren Schwimmkräne, die die Grundlage zu diesem Verfahren bilden, angegeben sowie deren Wirkungsweise erläutert. Die Grossfertigteil-Bauweise wird vom Standpunkt der Sicherheit aus diskutiert, und zum Schluss wird die Montage der Arakawa-Küste Brücke als ein typisches Beispiel dieser Bauweise beschrieben.

Membrane-Skin and Cable-Net Cooling Towers

Tours de refroidissement, avec une couverture d'aluminium posée sur un réseau de câbles

Kühltürme mit Membran- und Seilnetzmantel

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General description and load bearing behaviour:

The main feature of the new type natural draught cooling tower, presented here, is its prestressed membrane- or cable-net skin, replacing the reinforced concrete shell as with conventional towers (Fig. 1).

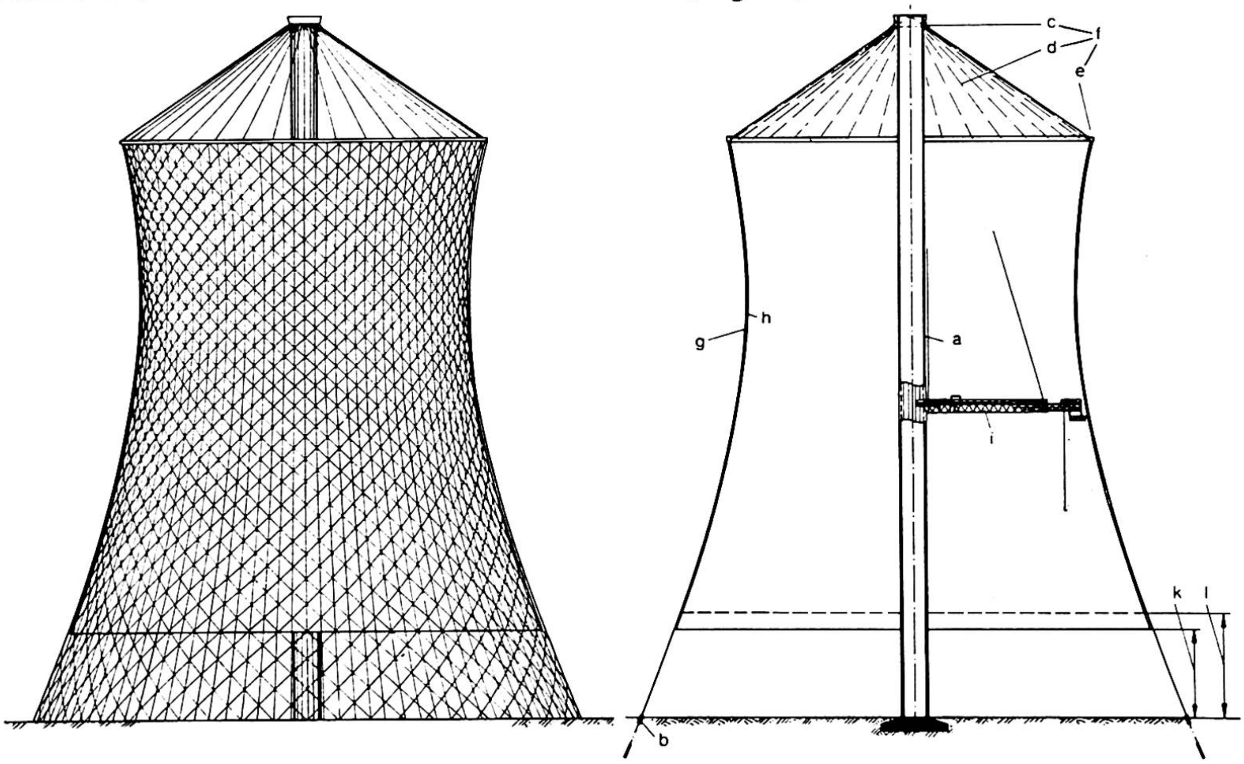


Fig. 1 a Mast, b Foundation ring with soil anchors, c Hubring, d Spokes, e Compression ring, f Spoke-wheel, g cable-net mantle, h Cladding, i Telescopic crane, k Air-intake opening, l Cooling elements resp. trickle plant

The one-sheet hyperboloid, commonly used for cooling towers, meets already excellently the shape requirements of negative curvature for prestressing. In a correspondingly cut membrane, closed in its circular direction, the prestressing forces required are to be applied only from its upper and lower borders to produce tensile stresses in the membrane at any point and in any direction. For the membrane always to act in tension, the amount of tension from prestress is required to be larger than the principle compressive stresses under any loading condition (Fig. 2).

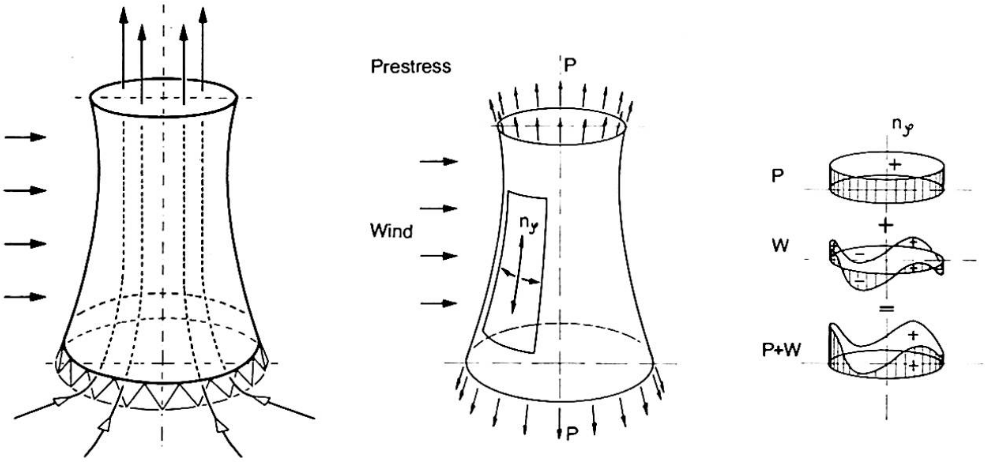


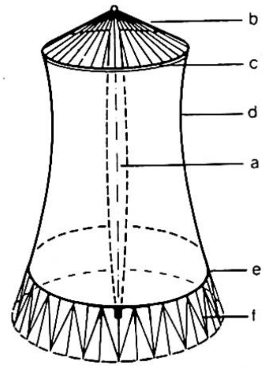
Fig. 2

Such a prestressed membrane skin behaves as an ideal membrane shell. When compared to a non-prestressed, and therefore necessarily thick shell with bending stiffness, as in the case of the conventional concrete cooling towers, it has certain decisive advantages:

- Local wind gusts will not cause bending stresses in the wall, since it has no bending stiffness. They will be distributed in the prestressed membrane through small deformations and minor additional direct stresses.

- There will be no stability or buckling problems, since the prestressed membrane never acts in compression. The tensile strength of its material may, therefore, be increased to the technically possible limits and be fully utilized. For this reason, materials such as high tensile steel sheets, fabric or cable nets are particularly suitable.

In order to introduce the prestressing forces into the membrane and to stiffen it against non-extensional deformations, it is provided with a foundation ring at the base and a compression ring at the top end (Fig. 1 + 3).
The compression ring is suspended by inclined radial ropes, similar to the ring in a spoke-wheel, from the top of a mast.



a Mast
b Spokes
c Compression ring
d Membrane mantle
e Intermediate ring
f Rod or cable truss

Fig. 3

The mast is placed at the center of the tower and acts under pure compression. The load bearing behaviour of the membrane shell will also not change if it is raised above ground level to a height determined by the air-intake requirements of the cooling system, provided that the membrane is replaced by a triangular mesh truss formed by ropes or bars.

The spoke-wheel, proposed for the suspension and the prestressing of the membrane shell, acts additionally as a perfect stiffening slab. By it the stresses in the shell will be much more uniformly distributed as compared to a shell without a stiffener at its top.

For cooling towers with a large height/width ratio it may be advisable to provide, in addition to the spoke-wheel at the top, intermediate horizontal spoke-wheels at various levels to approach more and more the optimum straight-line stress distribution of the fully stiffened shell. Since the size of the membrane forces is responsible for the potential energy of the tower, and as the maximum compressive forces in the skin mainly determine the amount of pre-stress required and, therefore, also the compressive force in the mast, the effect of these spoke-wheels is directly reflected in the costs of such cooling towers.

Structural design and erection of a first cable-net cooling tower:

At Schmehausen, West Germany, the first cable-net cooling tower for a nuclear power plant is presently (end of 1975) nearing completion. Its diameter at the base is 141 meters and the diameter of its compression ring at the top 91 meters. The mast is 180 meters high, with the height of the compression ring being 146 meters above ground level (Fig. 4).

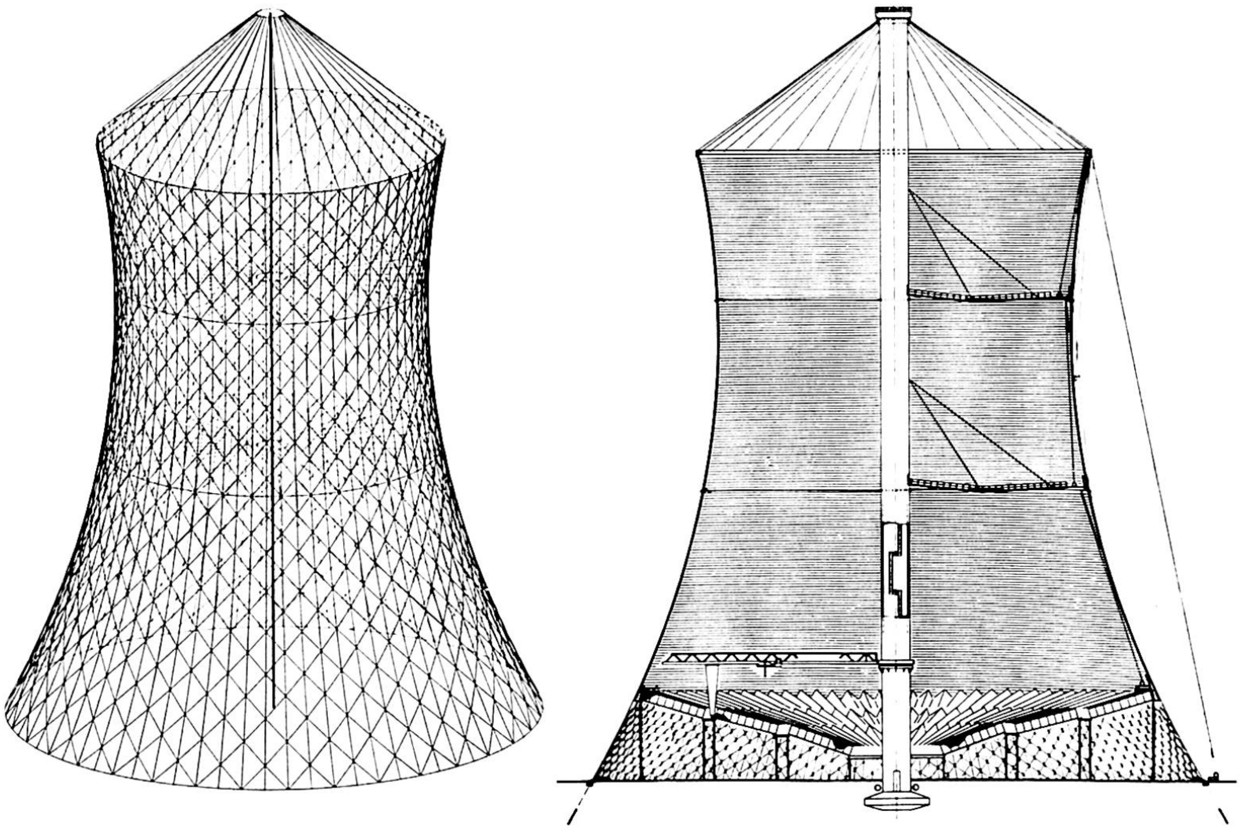


Fig. 4 The cable-net tower for the nuclear power plant at Uentrop/Schmehausen of the HKG im VEW-Kraftwerk Westfalen.

Instead of making direct use of a membrane as described above (Fig. 3), its skin consists of a prestressed cable-net which is further covered by an air-tight cladding. The cable net, with respect to its load bearing behaviour, must necessarily have a triangular mesh form, in order to act as a membrane. The arrangement of the cables was chosen in such a way, that the total net with its 46,000 m^2 can be prefabricated out of only two different ropes, as all diagonal ropes on the one hand and all meridian ropes on the other are exactly equal.

Since the central mast only acts under compression, it is made out of reinforced concrete. It was erected simultaneously with the casting of the circular ring foundation for the anchorage of the cable net (Fig. 5). This reinforced concrete ring is anchored into the ground by prestressed soil anchors.

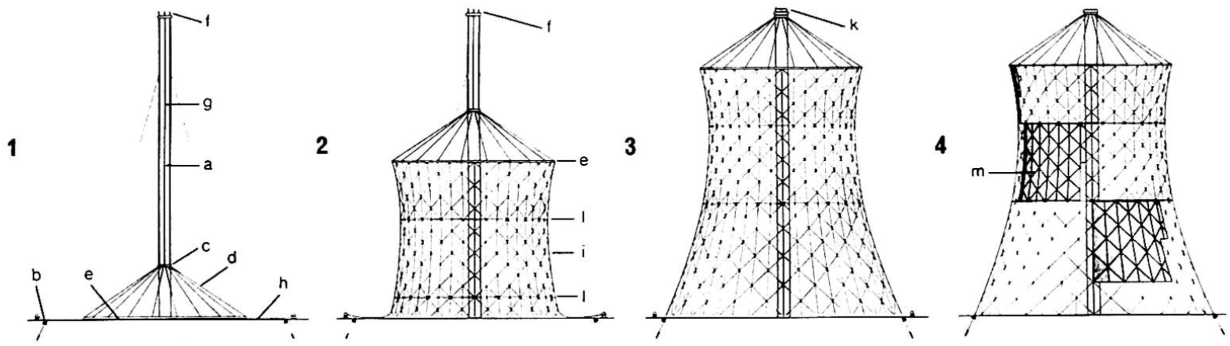


Fig. 5 a Mast, b Foundation ring with soil anchors, c Hubring, d Spokes, e Compression ring, f Lifting device, g Lifting ropes, h Net ropes, i Cable-net mantle, k Prestressing device, l Spoke-wheels, m Cladding.

Next the outer compression ring of the upper spoke-wheel and its central hub-cum-lifting ring were assembled on the ground. Both rings are steel hollow-box sections, 80 x 120 cm and 60 x 100 cm respectively (Fig. 6).

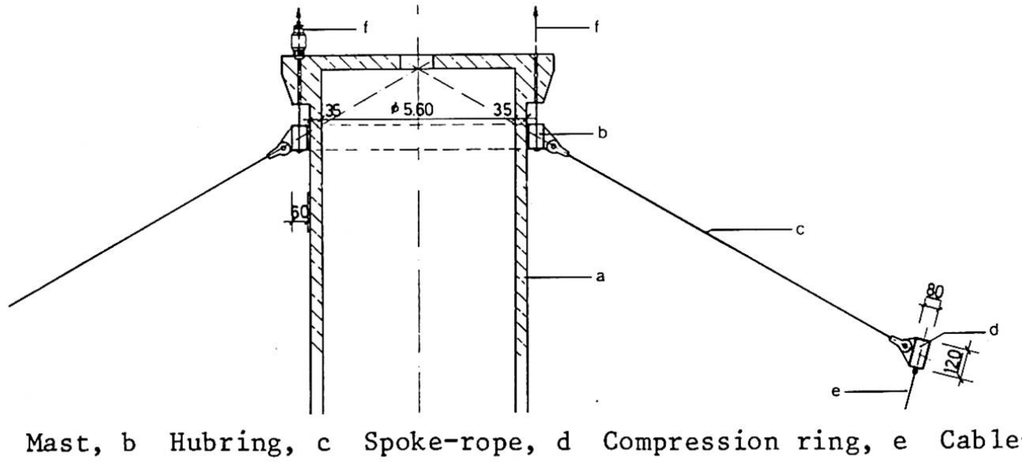
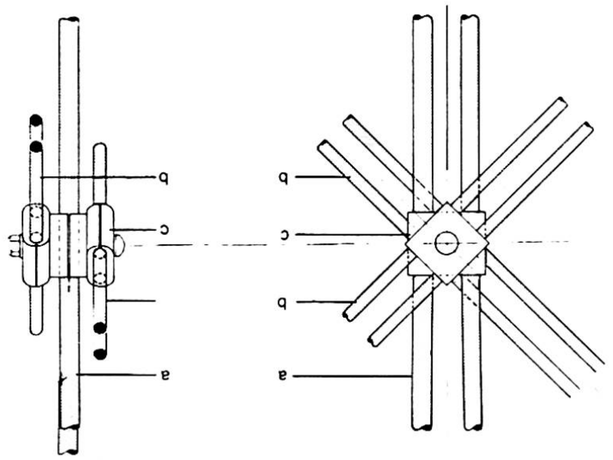


Fig. 6 a Mast, b Hubring, c Spoke-rope, d Compression ring, e Cable-net, f Lifting resp. prestressing device.

The 36 spokes, fabricated using 78 mm diameter locked-coil ropes, were connected to these two rings. In the meantime, at the top of the mast a lifting device was installed from which the central hubring was raised by three lifting ropes. The cable-net was then attached to the compression ring and continuously assembled whilst the hubring and consequently the spoke-wheel was lifted. For this purpose the type of net ropes, already developed for the Olympic roofs in Munich, were most suitable. These ropes consist of two parallel strands with aluminium clamps, press-fitted in the fabrication workshop (Fig. 7).



a Vertical or meridian ropes;
d = 25 mm
Strand from 37 Alumoweld wires

b Diagonal ropes;
d = 20 mm
Strand from 19 Alumoweld wires

c Aluminium press-fitted clamps

Fig. 7

They guarantee an exact pattern of the net through a continuously equal distance between the corresponding knots. For assembling the net on the site, there is only one bolt at each knot required for the connection of the three layers of the rope. To guarantee long-life corrosion protection, the strands are spun from so-called alumoweld wires, which are steel wires, covered with an aluminium coating of as much as 25 % of the total section.

After the lifting process was finished, the cable-net was connected with the ring foundation (Fig. 5). Next the hubring at the top of the mast was further lifted until the prestressing force required was reached. According to its pattern, the cable-net then assumed its exact geometry.

In accordance with the principles mentioned before, the Schmehausen tower is being constructed with two additional horizontal spoke-wheels (Fig. 4 + 5). Each consists of an outer compression ring on the net surface and an inner tension ring encircling but not touching the mast. Both rings are connected by 36 post-tensioned radial ropes of 32 mm diameter. They were also assembled at ground level, simultaneously with the assembling of the cable-net, and lifted together with it. After prestressing the cable-net and connecting the hubring with the top of the mast and thus finishing the structural construction of the tower, the cladding is attached onto the inside of the net.

The cladding, in this case, consists of corrugated aluminium sheets, which are bolted to the knots of the cable-net and are joined in such a way that they are able to follow the deformations of the cable-net under wind load. The net on the outside yields a surface roughness which minimizes the wind suction at the flanks of the tower, as could be shown by wind tunnel tests.

Outlook:

Since there is almost no limitation to the size of membrane- or cable-net cooling towers, as it is the case with concrete towers, this new type can be expected to become most advantageous for the future large and very large cooling towers. It will with increasing size become comparatively easy to erect and more economical, mainly if the required width is to be large against the height (Fig. 8).

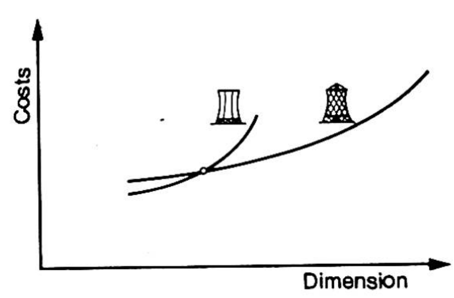


Fig. 8

Under seismic conditions it will in any case be superior to the conventional type, because, due to its small mass, even the most severe earthquakes cause less forces than wind. It should further be mentioned that this type of cooling tower is not at all sensitive to foundation settlements. If in mining areas very large settlements are expected, turn-buckles can be provided in the ropes along the foundation for easy adjustments.

This type of structure permits such large dimensions for cooling towers where the area inside the tower yields enough space for a whole power plant. The mast may then serve, at the same time, as a chimney. It is also possible to design the cable-net as an anti-aircraft net for any nuclear power plants, built inside the cable-net cooling tower.

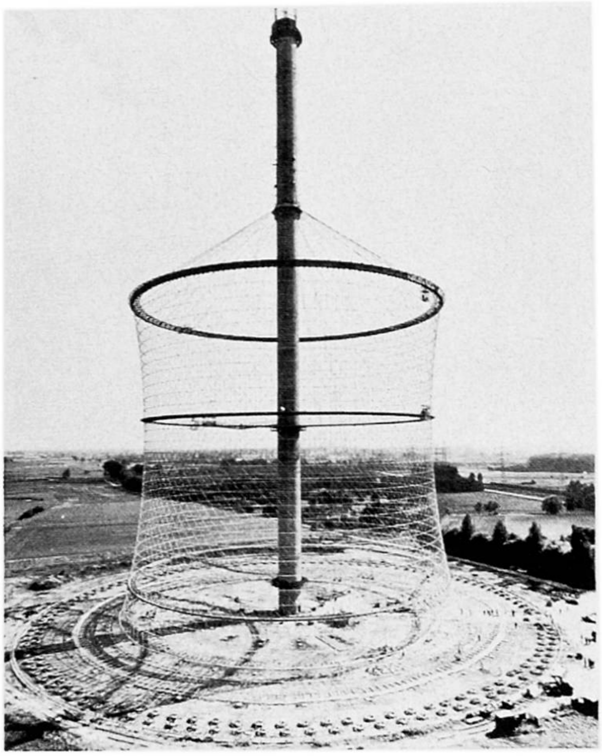


Fig. 9 The cable-net tower for the nuclear power plant at Uentrop/Schmehausen. Completed works up to Sept. 75 (see Fig. 5, construction phase 2).

The cooling tower at Schmehausen (Fig. 4 + 9), as described in this paper, is a joint development of the Balcke-Dürr AG, Bochum, and Leonhardt + Andrä, Consulting Engineers, Stuttgart, with J. Noesgen being the main collaborator of the authors. Balcke-Dürr/GEA, Bochum, are the general contractors and Krupp Industrie- und Stahlbau, Goddelau, the contractors for the cable-net tower. The computer program was developed by D. Scharpf of the RIB, Stuttgart.

SUMMARY

A new type of natural draught cooling tower is presented, which is mainly of advantage, if either one of the following conditions is required: large dimensions (specially large width to height ratio), seismic conditions and soil settlements.

RESUME

Une nouvelle tour de refroidissement est décrite, qui est surtout avantageuse si l'une des exigences suivantes est remplie: grandes dimensions (particulièrement grand diamètre par rapport à la hauteur), construction en zone sismique ou sur des terrains exposés aux tassements.

ZUSAMMENFASSUNG

Ein neuartiger Naturkühlturm wird beschrieben, der vor allem dann vorteilhaft ist, wenn eine der folgenden Forderungen gestellt wird: grosse Abmessungen (besonders grosser Durchmesser gegenüber der Höhe), Bau in Erdbebengebieten oder setzungsempfindliche Böden.