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

Entwurfsgrundlagen und Entscheidungskriterien für Tragwerke

Design Philosophy and Decision Processes for Structures

I a

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Planning of Structures and its Relationship with Construction Methods

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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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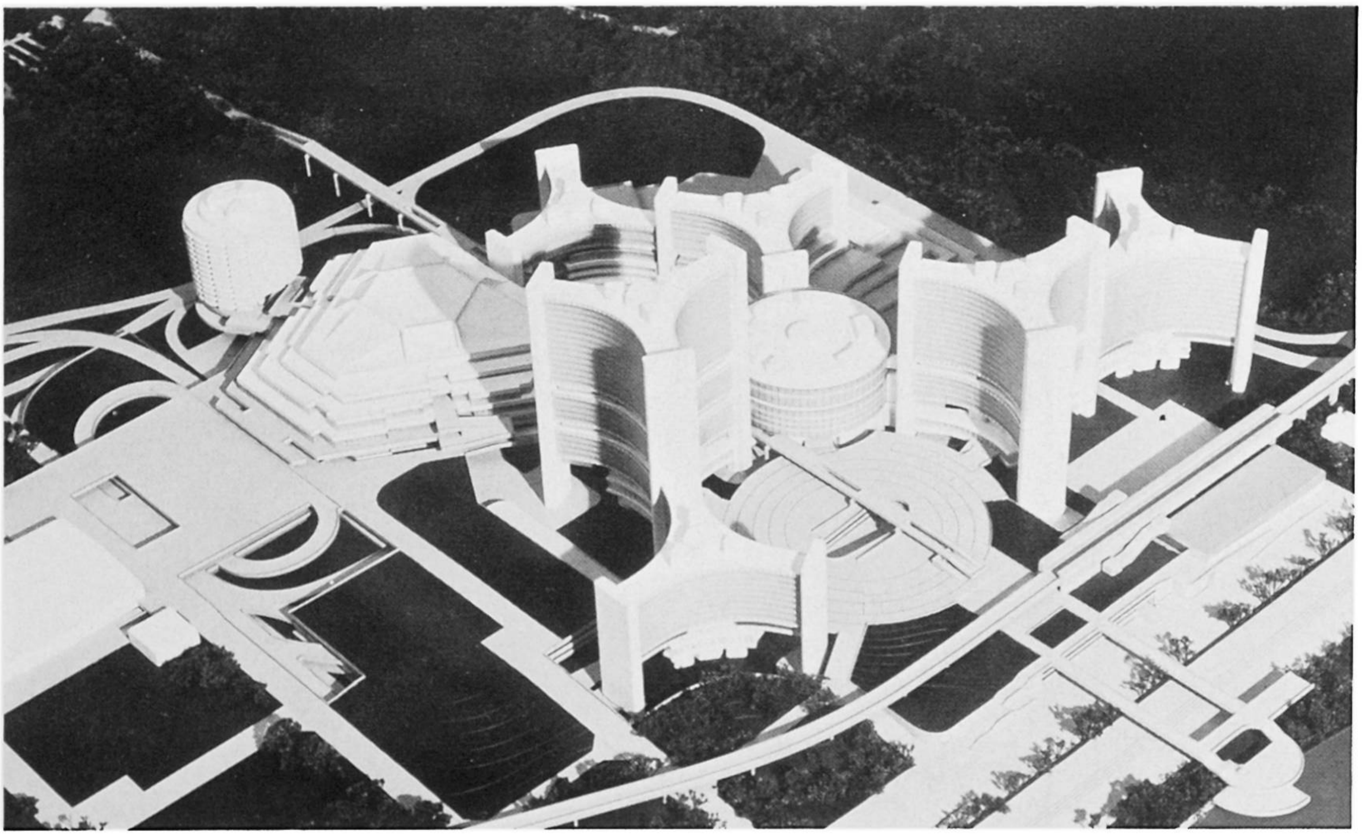
1. Einleitung und Aufgabenstellung

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.



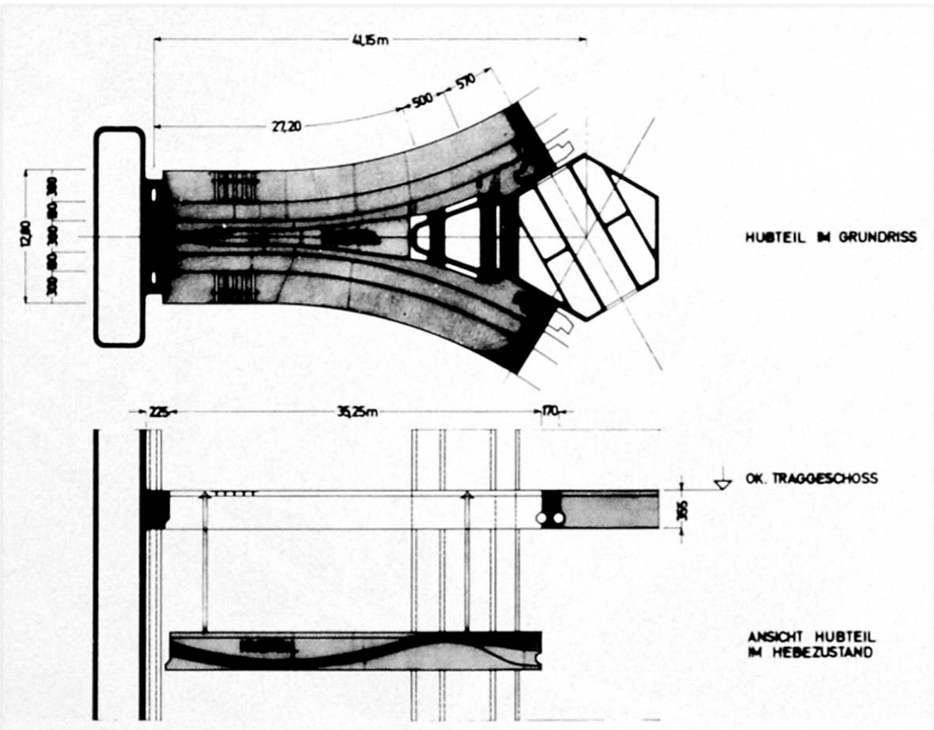
Die Planung dieser Objekte, die sich im Grundriß, in der Höhenentwicklung und im Querschnitt stark von üblichen Hochhausbauten unterscheiden, war gekennzeichnet durch einige neuzeitliche Ausführungsmethoden, die mit der besonderen Architektur starken Einfluß auf die Konstruktion ausübten.

Für den statischen Entwurf ergaben sich somit folgende allgemeine Vorbedingungen:

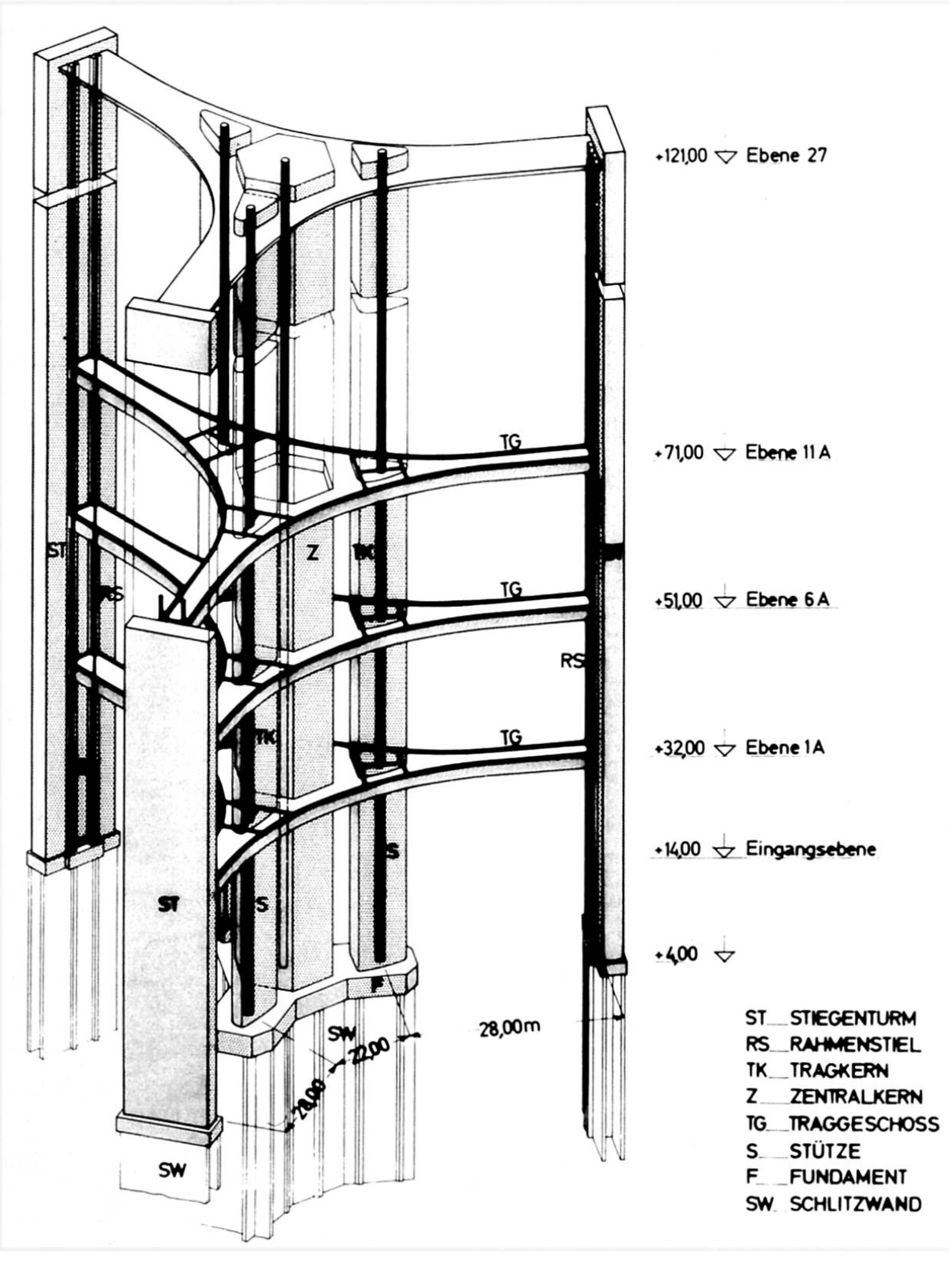
- Errichtung und Vorziehen aller lotrechten Tragglieder in Gleitbauweise.
- Gerüstlose Herstellung der die Gebäudemassen aufnehmenden Traggeschoße als dreiteilige Großfertigteile am Boden und hydraulisches Heben derselben in die endgültige Lage im Bauwerk (Hubgewicht der Einzelteile 1400 t).
- Ausbau 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.



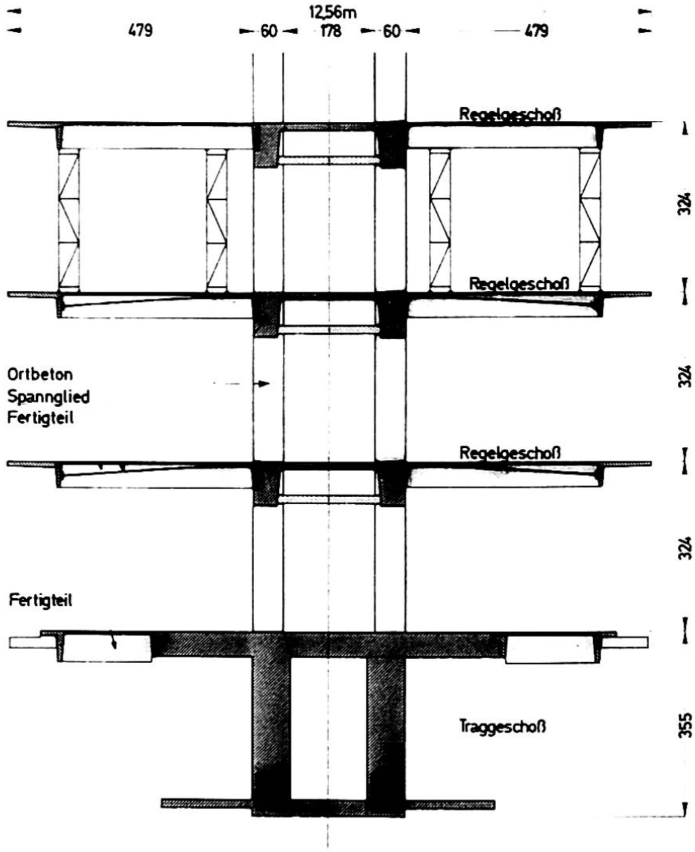
Von ganz wesentlicher Bedeutung war weiters die Erzielung eines genügend steifen Systems zur Ableitung der Seitenkräfte infolge Windwirkung und etwaiger Erdbebenkräfte, bedingt durch die Lage des Baugeländes an den Ausläufern eines Erdbebengebietes. Die naheliegende Lösung mittels kräftiger Scheiben 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.

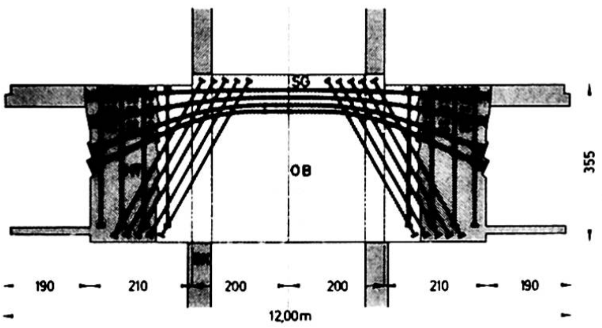


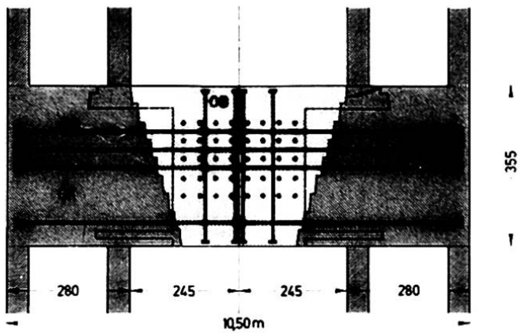
3. Einige konstruktive Besonderheiten

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.





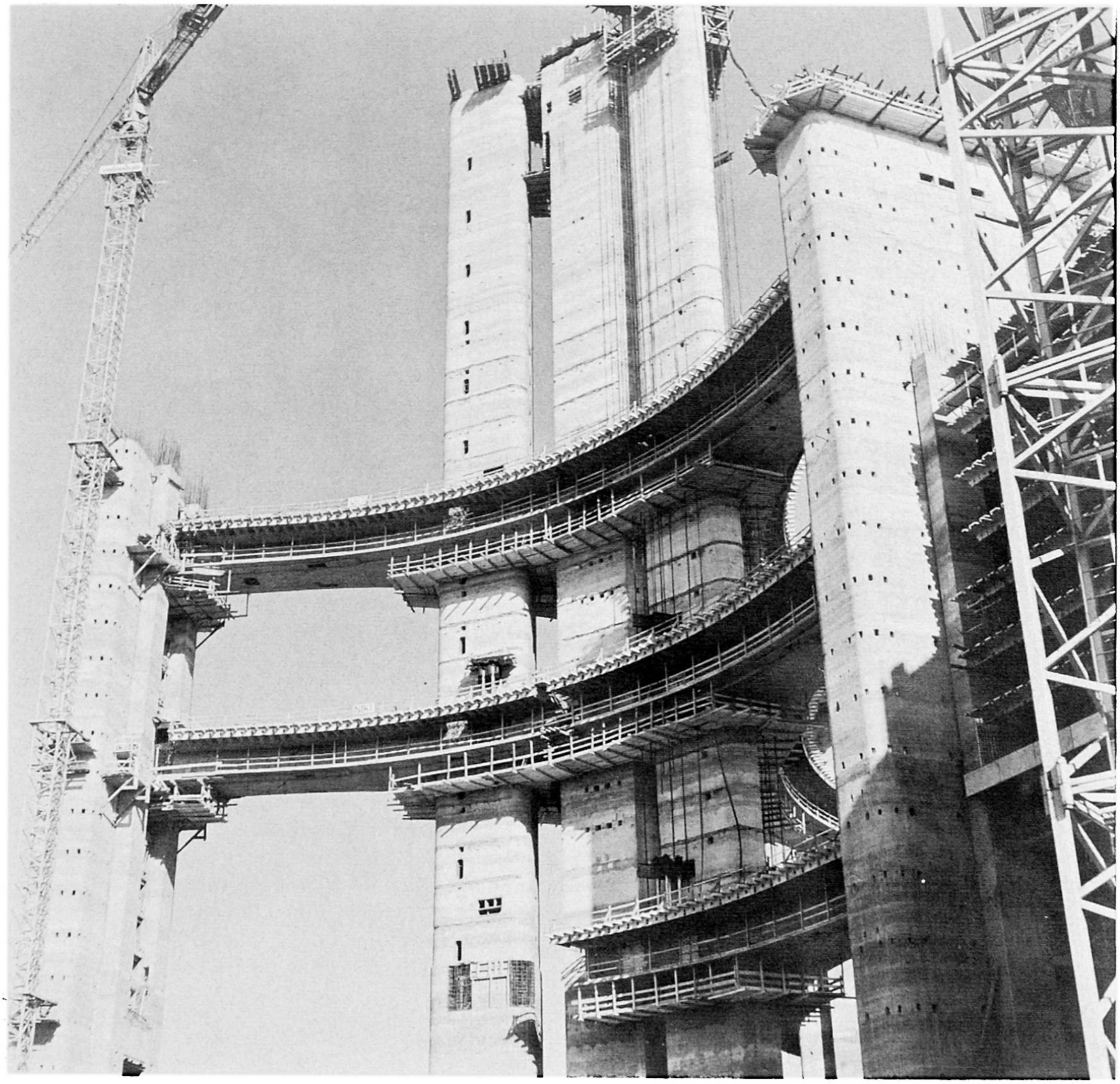


GK GLEITKONSTRUKTION
OB ORTBETON
HT HUBTEIL
SG SPANNGLIED

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

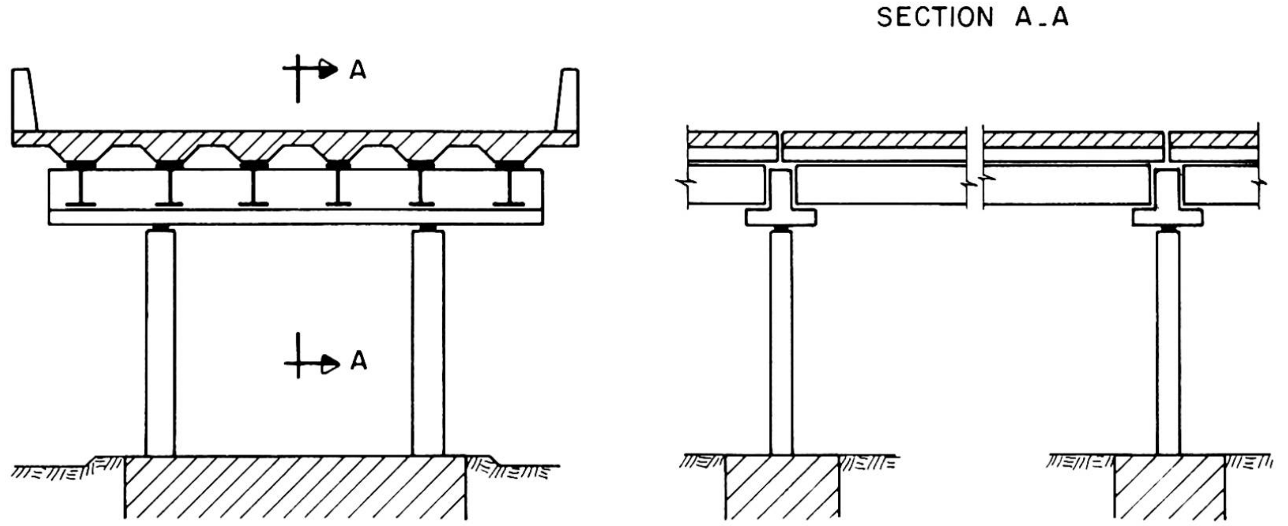
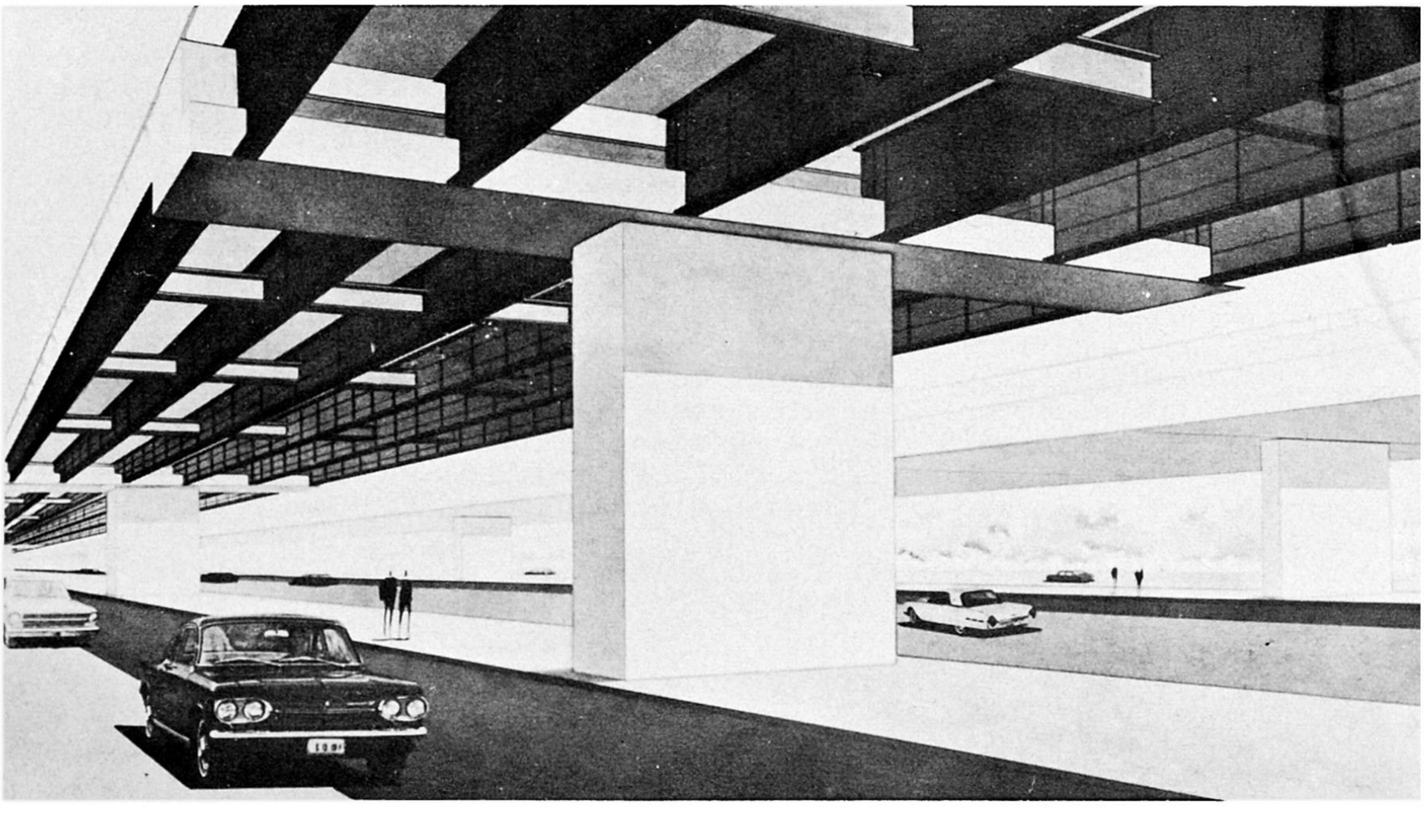
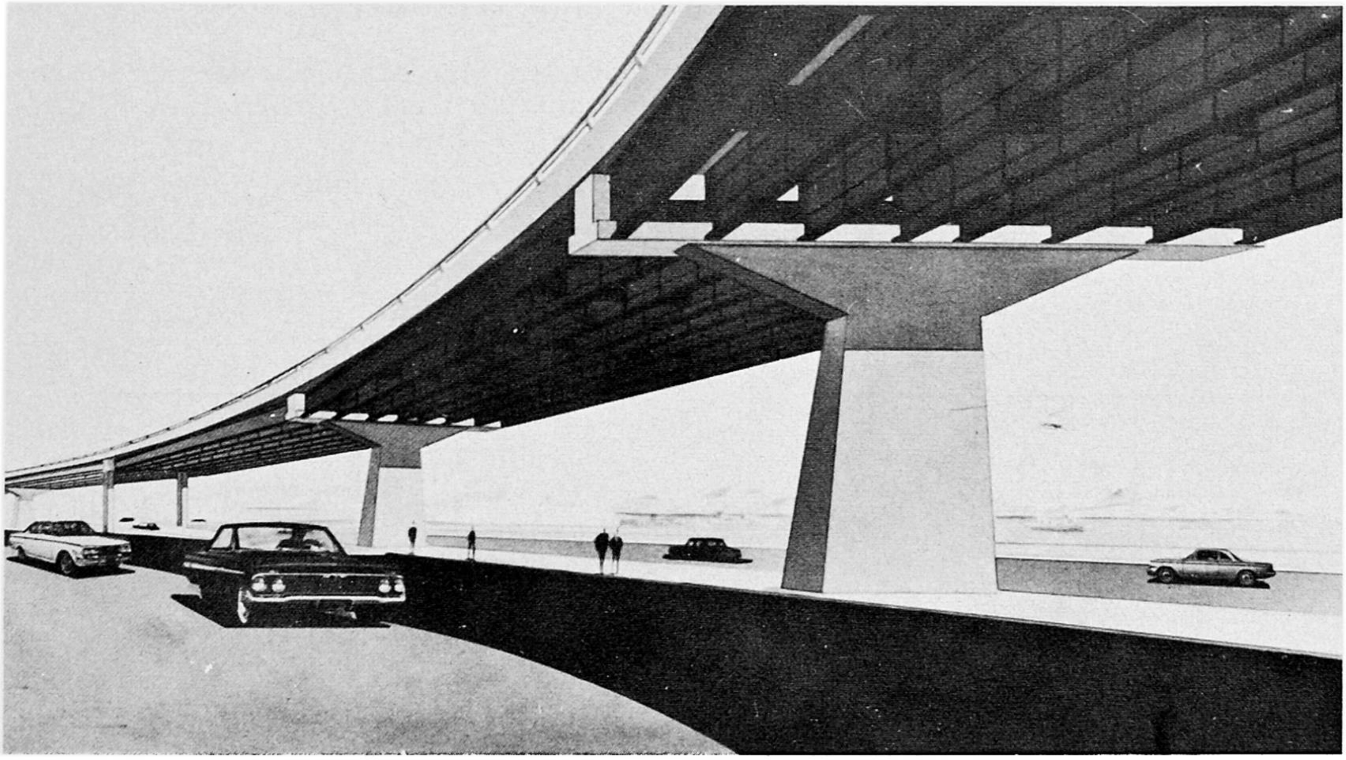


FIG. 1

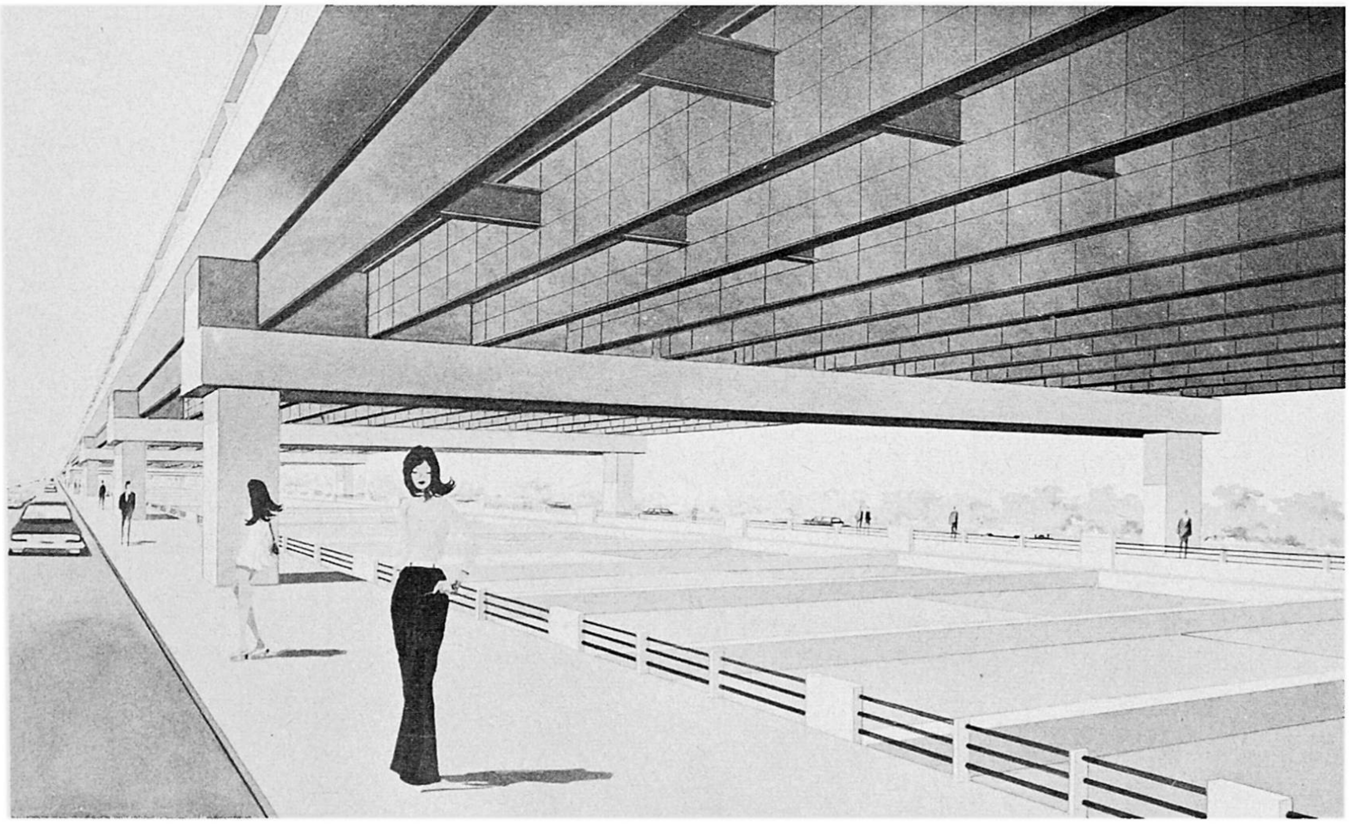
The photographs included illustrate additionally the aspects of the structural solution.





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).

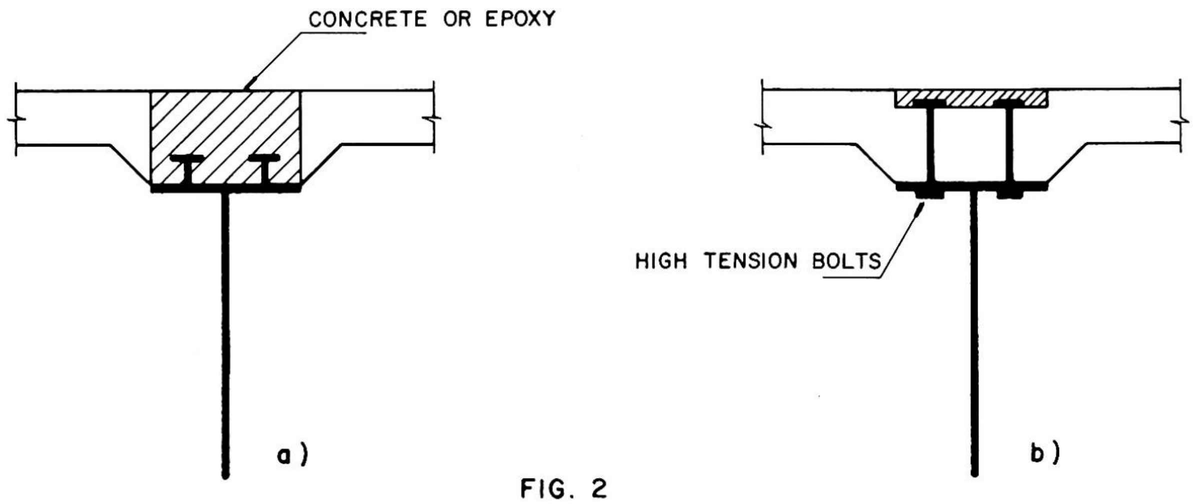


FIG. 2

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.

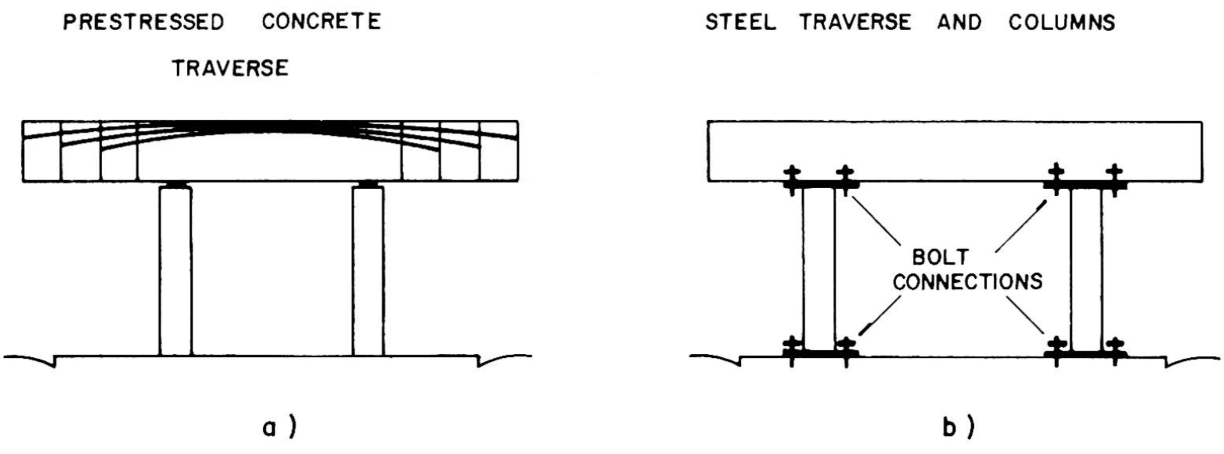


FIG. 3

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).

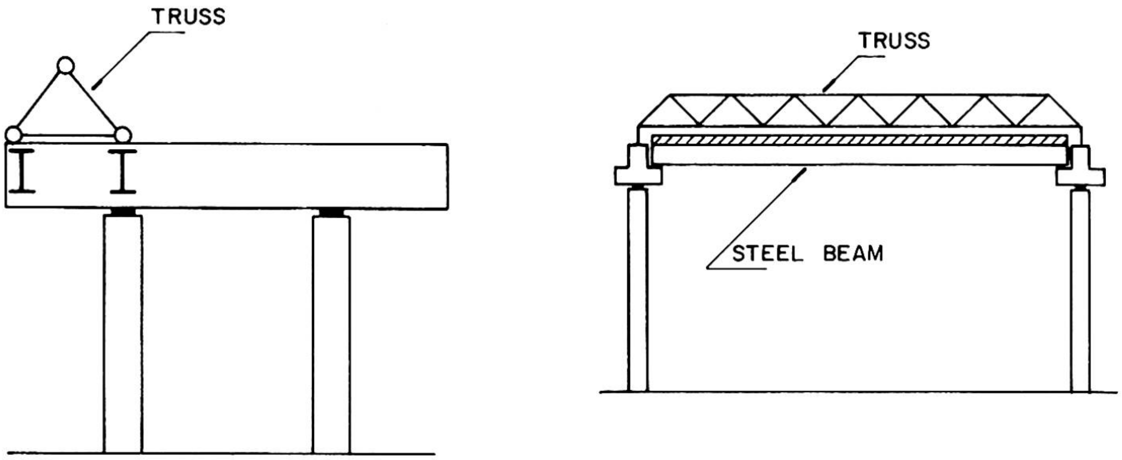


FIG. 4

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

3 - CONCLUDING REMARKS

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) Choix de solutions polyvalentes.

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) Division modulaire des projets et réalisations.

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.

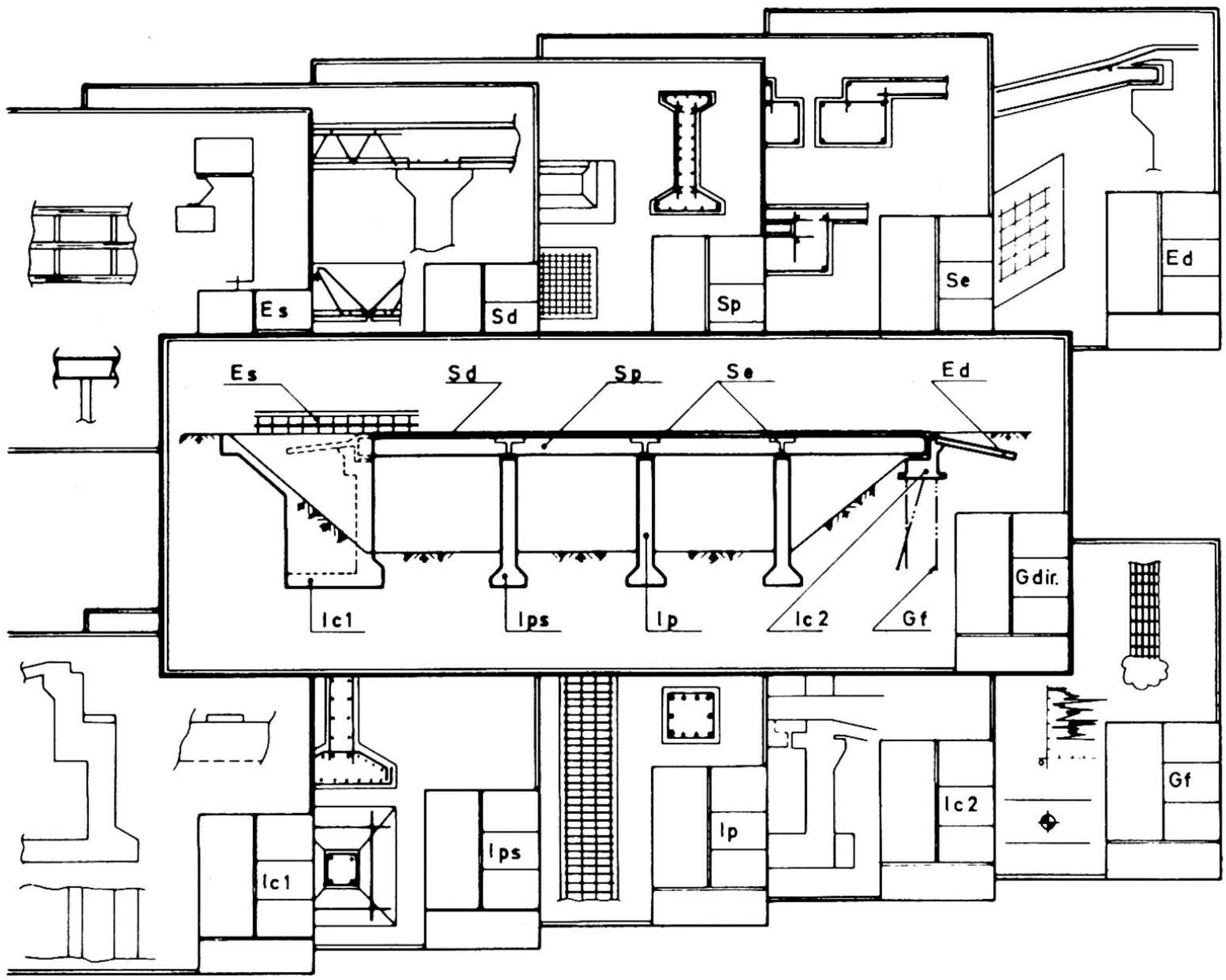
4) Caractère évolutif à donner à chaque partie et au schéma d'assemblage.

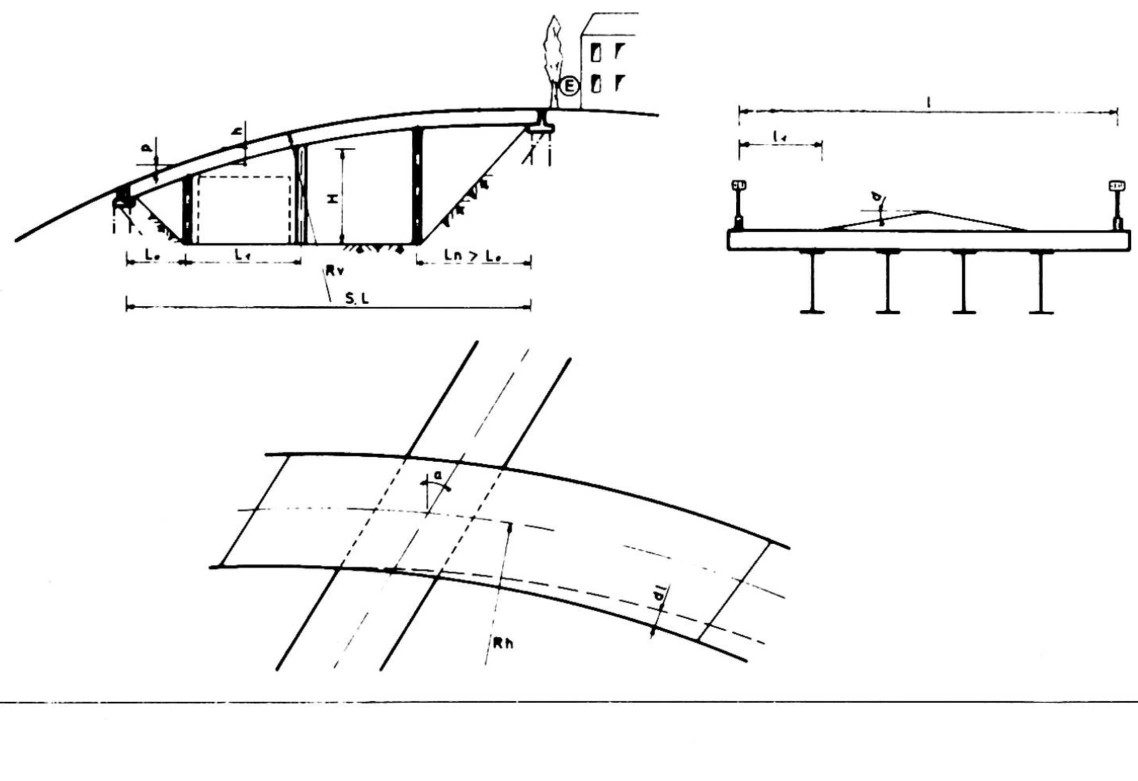
Pour éviter la stagnation, on doit veiller à ce que chaque partie et le schéma d'assemblage puissent être améliorés indépendamment chaque fois qu'une solution meilleure sera trouvée. En ce qui concerne l'aspect, la souplesse de la formule proposée permet également une diversité suffisante pour éviter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

C. Application pratique dans le cas de ponts.

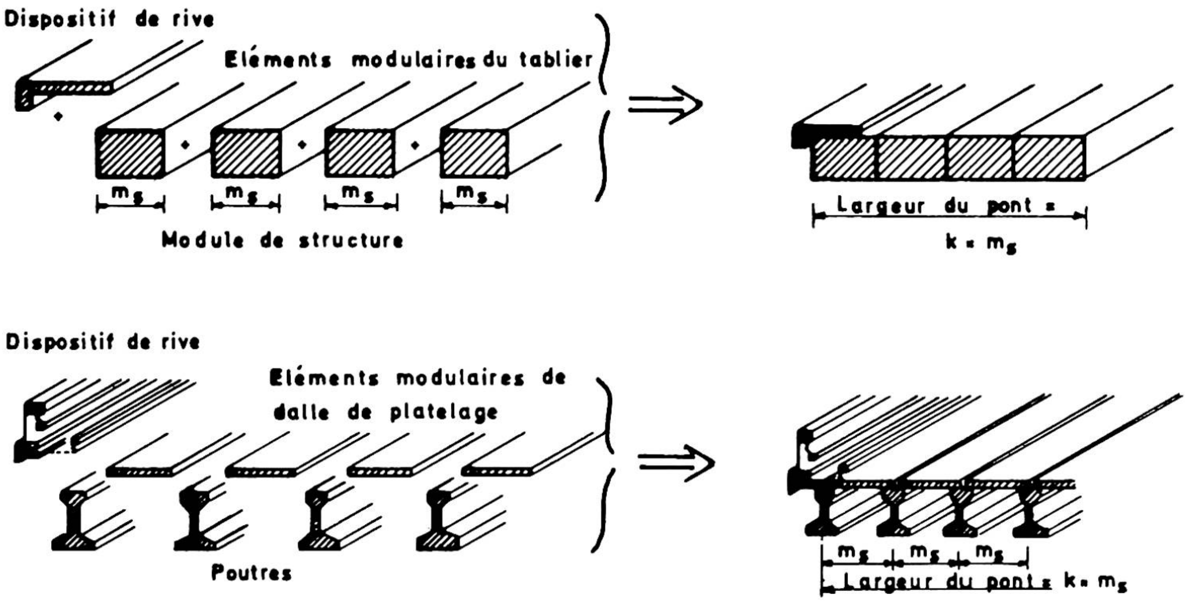
	Variables d'un pont donné	Moyen de réduire le nombre de variables et/ou le nombre de valeurs de celles-ci.
L.	portée	modulation (nombre de valeurs réduit)
S.L	longueur	modulation (nombre de valeurs réduit)
l.	largeur	structure indépendante de la largeur grâce à 1.modulation 2.faible entretoisement 3.suppression 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.différ.-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

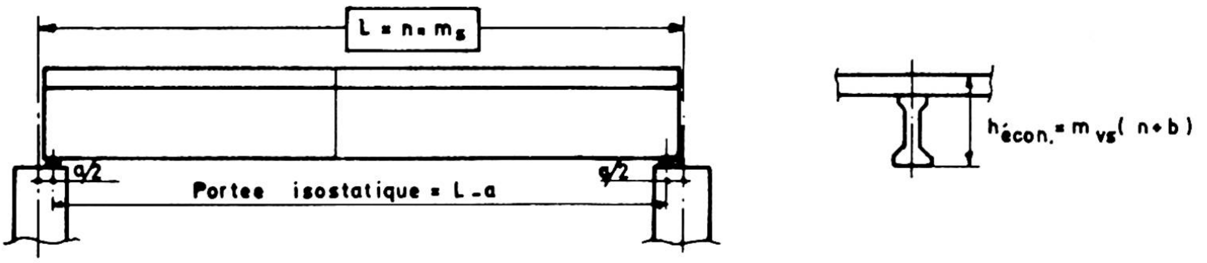




MODULATION TRANSVERSALE

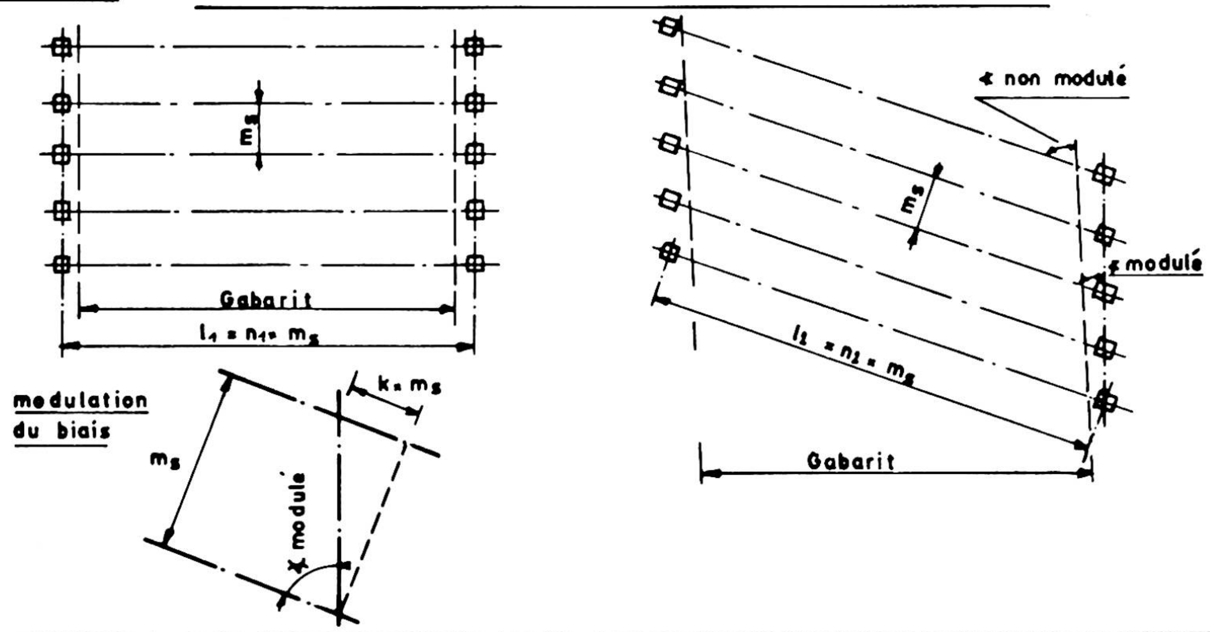


MODULATION LONGITUDINALE & VERTICALE



BIAIS

LIMITATION DE L'INFLUENCE DU BIAIS A UNE QUESTION DE PORTEE



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 differents, 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.

Ia

Design and Construction of the Hokawazu Bridge

Conception et réalisation du pont de Hokawazu

Entwurf und Ausführung der Hokawazu-Brücke

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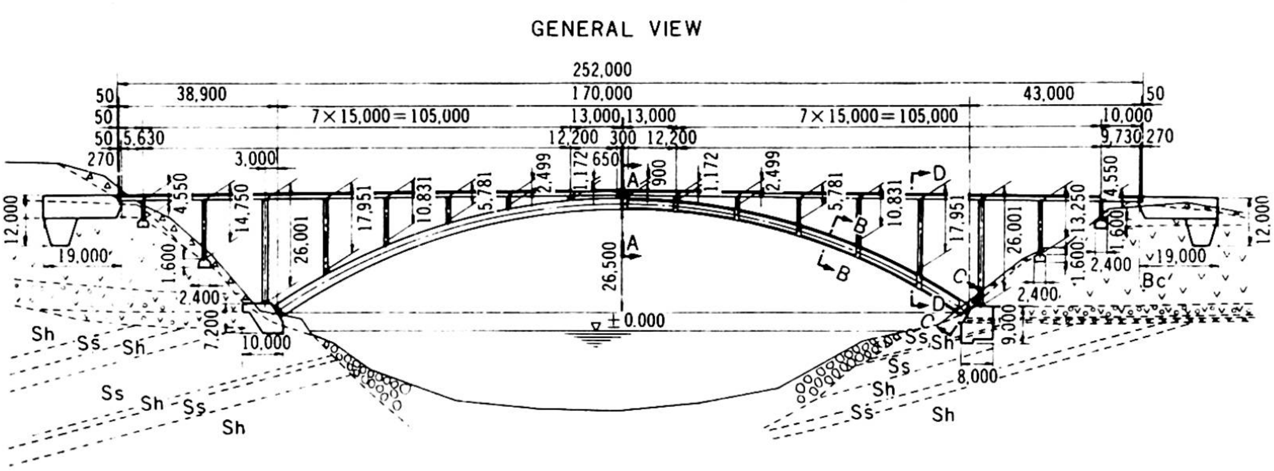
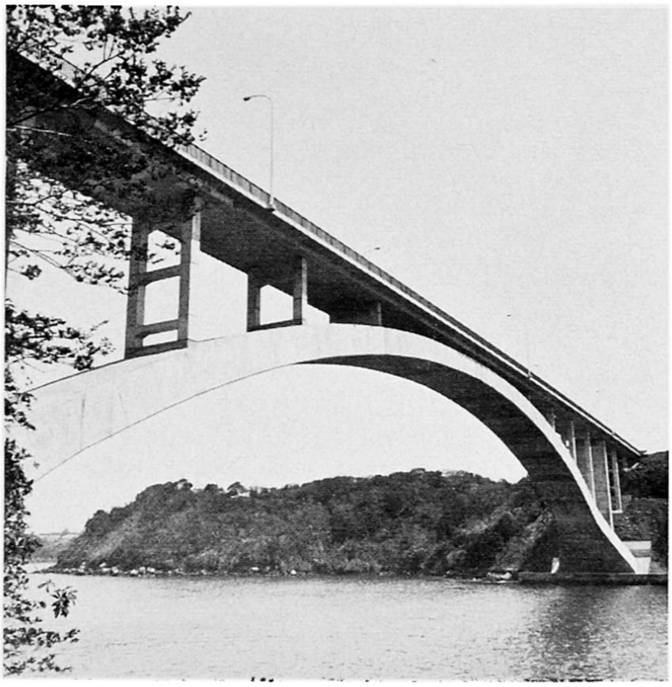
1. Preface

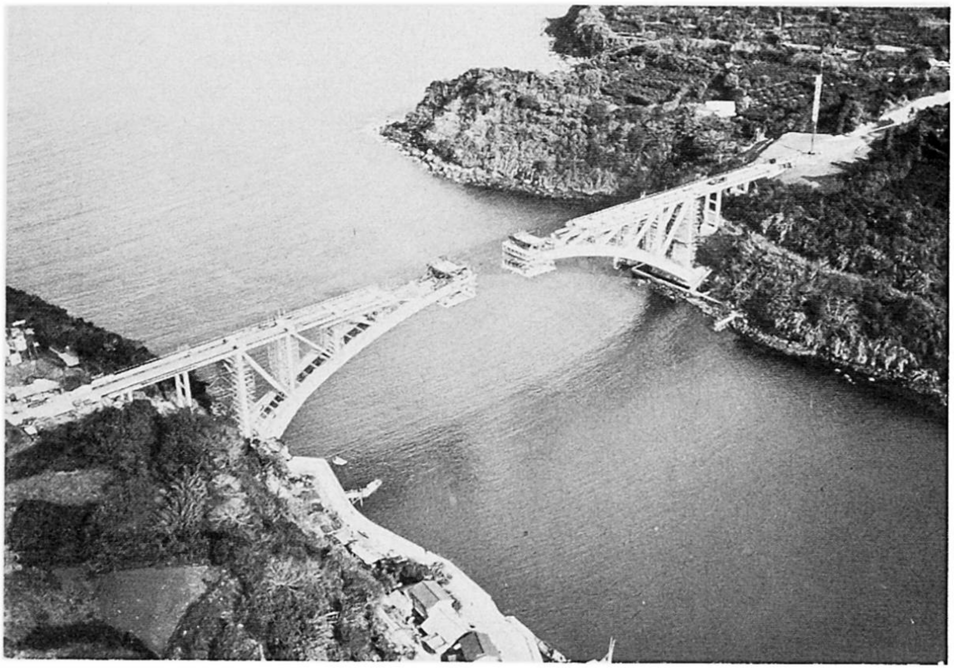
The National Highway Route 204 starts from Karatsu City in Saga Prefecture, goes along the Sea of Genkai, runs through Imari City in the same prefecture, and leads to Sasebo City in Nagasaki Prefecture, and the total length of the route reaches upto 154 km. The Hokawazu Creek made the route discontinuous 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.

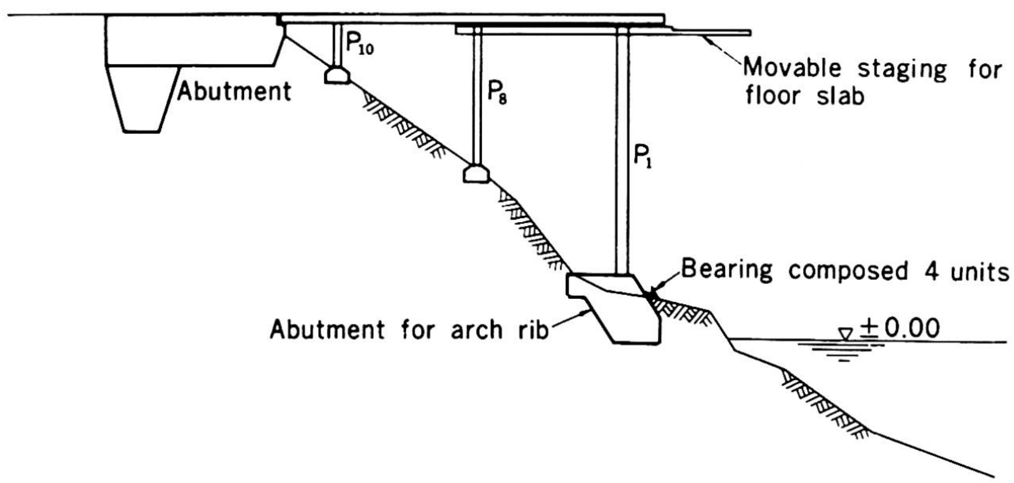
The dimensions of the abutments reach 20 m in width, 9 m in height and 8 m in length.

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.



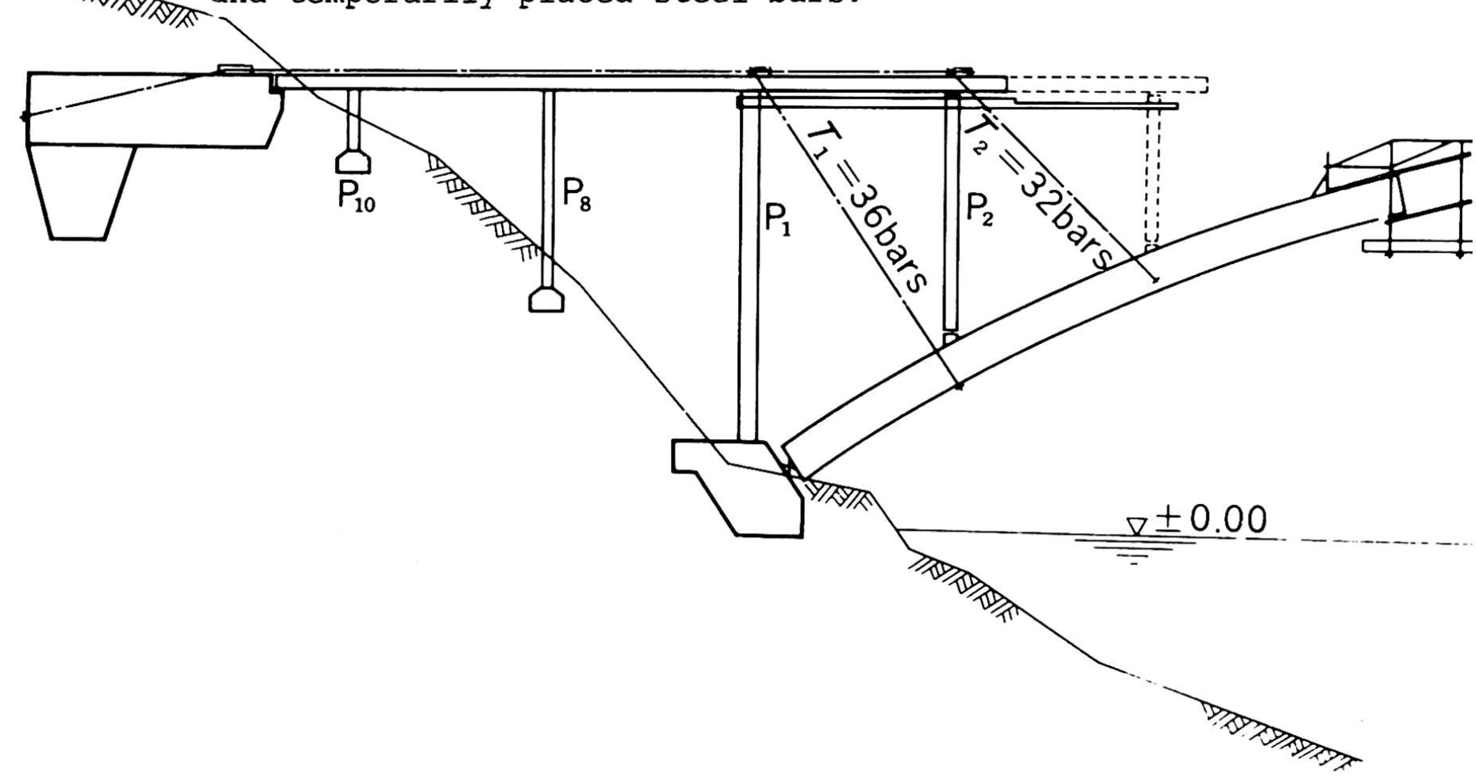
2) Construction of a part of the arch rib on suspended falseworks

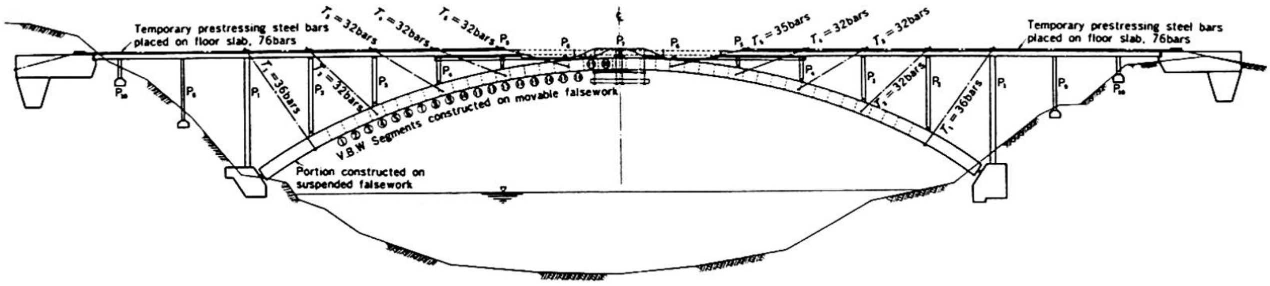
   i. Placement of steel panel girder falseworks suspended by 36 inclined prestressing steel bars, T_1, in the portions between P_1 and P_2.

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.

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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 construc-tion	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.

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.

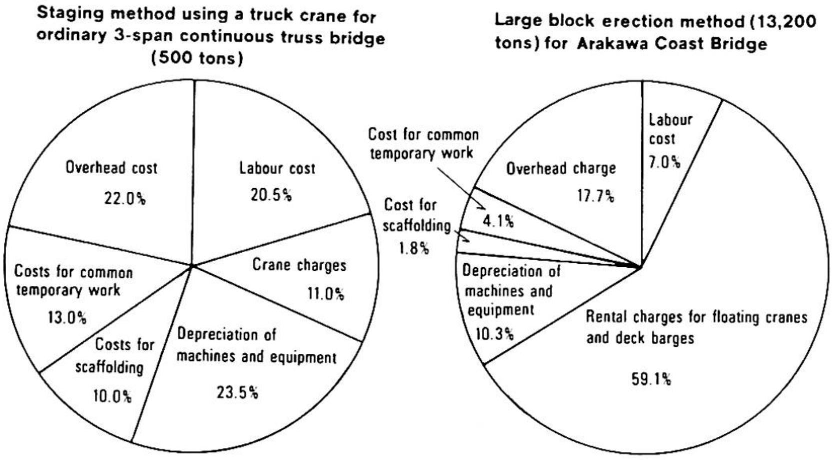

Fig. 1 Cost comparison between large block erection method and conventional erection method

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.

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.

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

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.

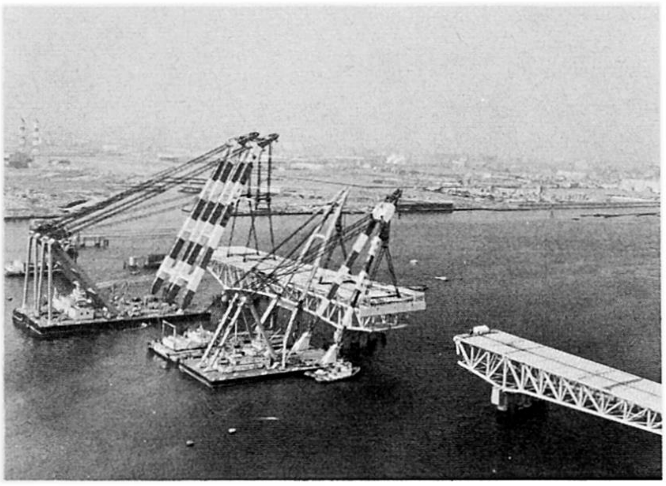

Fig. 3 Erection of the largest block (4,250 ton)

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).

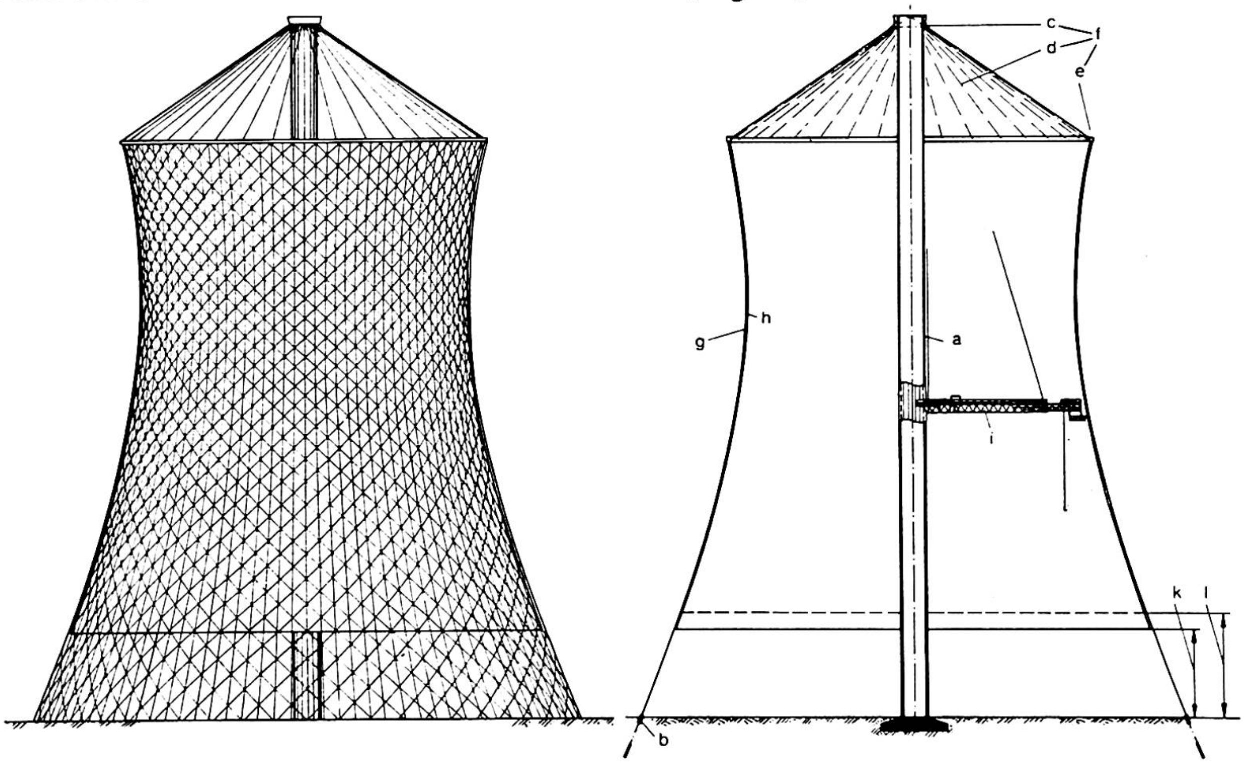


Fig. 1 a Mast, b Foundation ring with soil anchors, c Hubring, d Spokes, e Compression ring, f Spoke-wheel, g cable-net mantle, h Cladding, i Telescopic crane, k Air-intake opening, l Cooling elements resp. trickle plant

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).

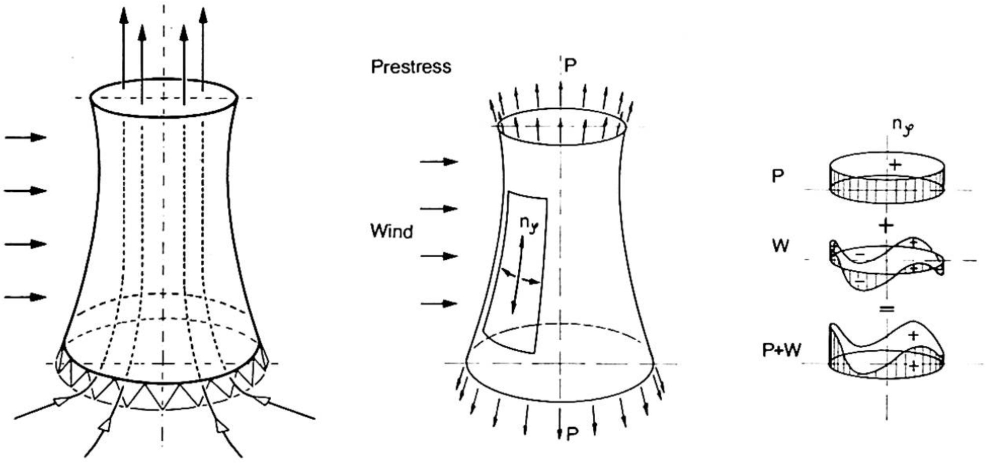


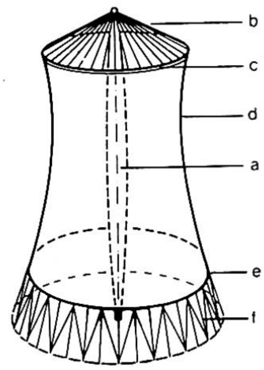
Fig. 2

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 prestress 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).

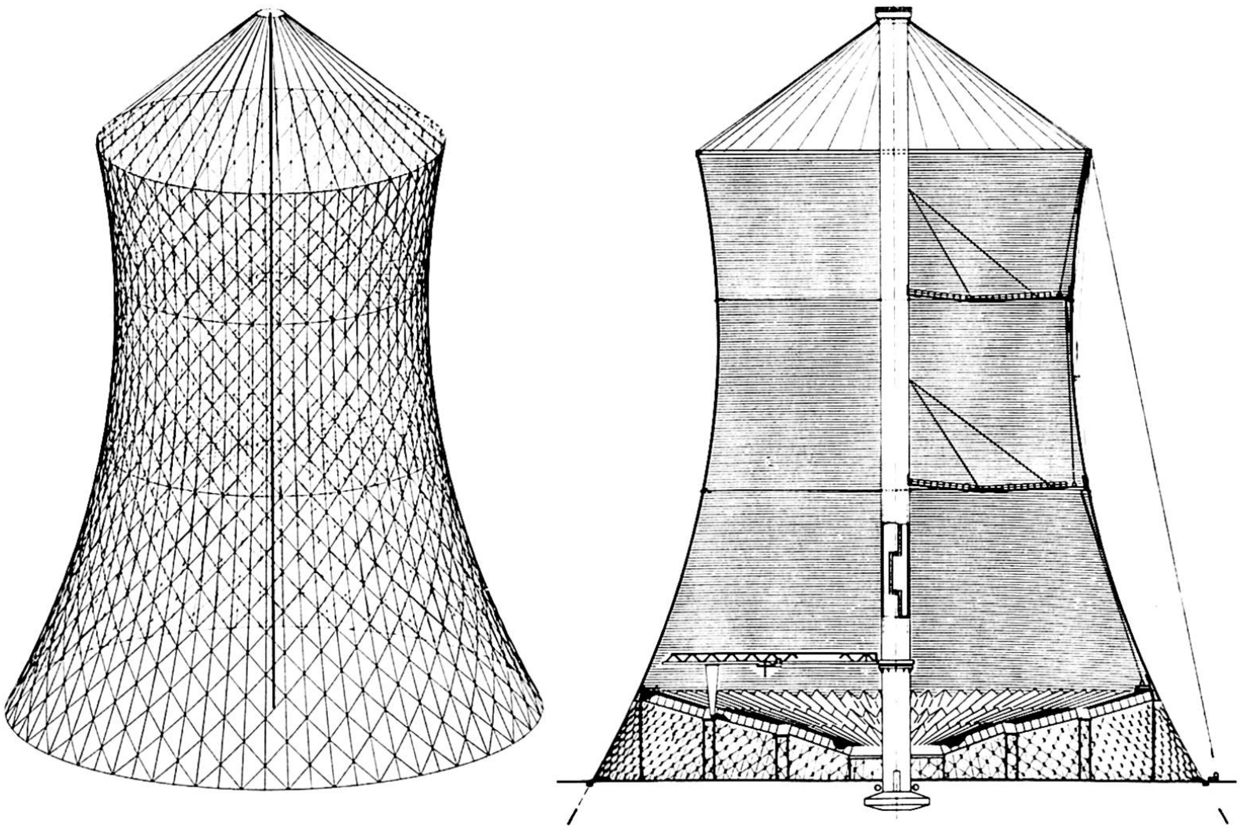


Fig. 4 The cable-net tower for the nuclear power plant at Uentrop/Schmehausen of the HKG im VEW-Kraftwerk Westfalen.

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.

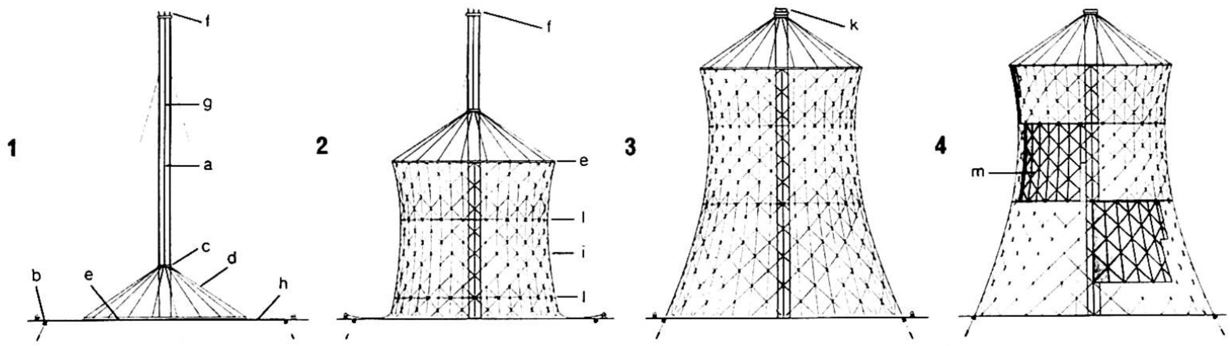


Fig. 5 a Mast, b Foundation ring with soil anchors, c Hubring, d Spokes, e Compression ring, f Lifting device, g Lifting ropes, h Net ropes, i Cable-net mantle, k Prestressing device, l Spoke-wheels, m Cladding.

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).

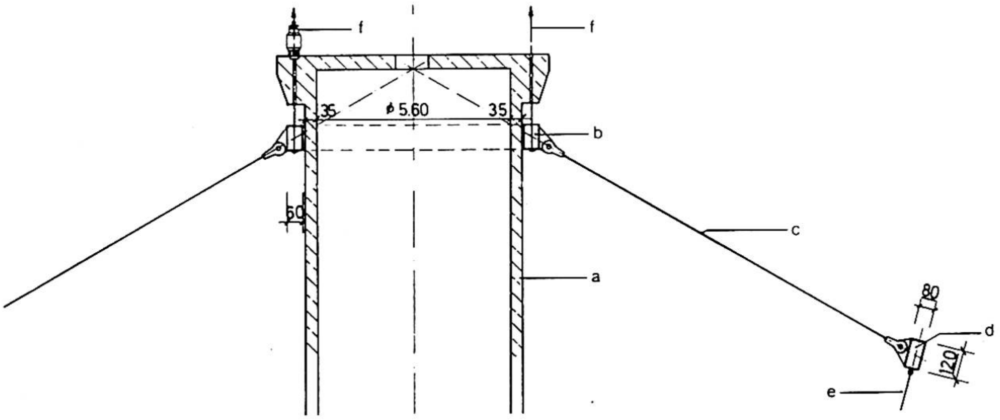
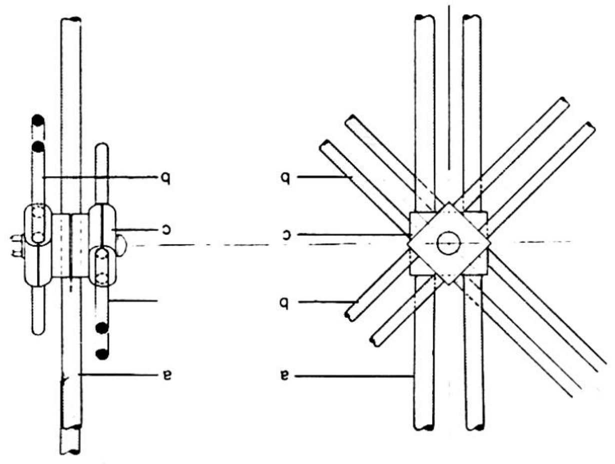


Fig. 6 a Mast, b Hubring, c Spoke-rope, d Compression ring, e Cable-net, f Lifting resp. prestressing device.

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).

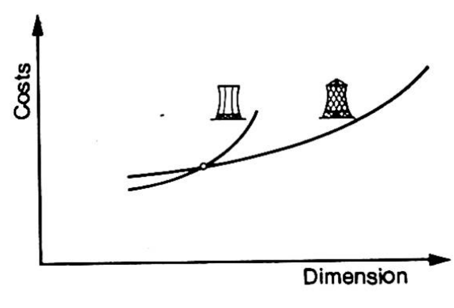


Fig. 8

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.

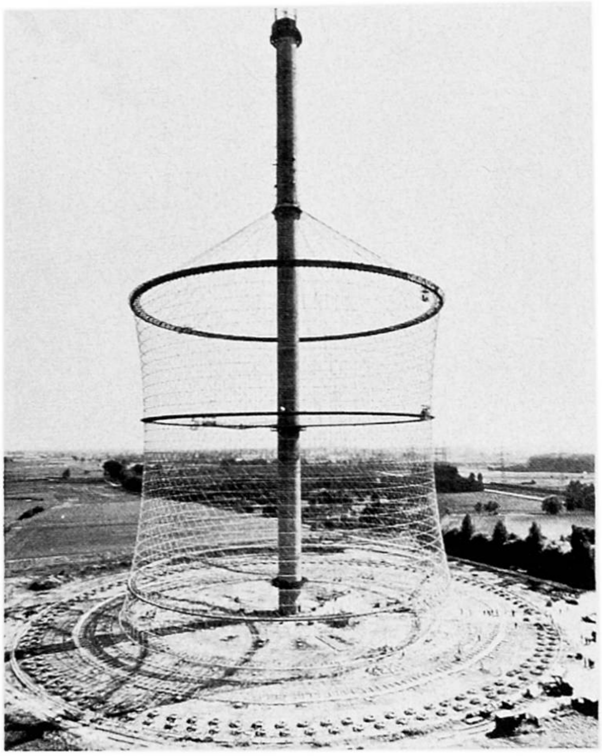


Fig. 9 The cable-net tower for the nuclear power plant at Uentrop/ Schmehausen. Completed works up to Sept. 75 (see Fig. 5, construction phase 2).

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.

Ib

Des exigences de la sécurité et du souci de l'économie dans l'étude et la construction

Sicherheits- und Wirtschaftlichkeits-Aspekte im Entwurf und in der Ausführung

Achievement of Safety and Economy in Design and Construction

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Creative Design based on Safety and Economy

Créativité et conception des structures, basée sur la sécurité et l'économie

Schöpferischer Bauentwurf gegründet auf Sicherheit und Wirtschaftlichkeit

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1. Introduction

Pozzi[1] in his paper deals generally with planning of structures. The structure as an element of the building construction is only briefly covered as a particular problem. The present paper deals with the design of a structure considering the main title of Theme (I) "Design Philosophy and Decision Process for Structures" bearing in mind also Theme (II) "Progress in Structural Optimisation".

The three papers in Theme (II) attempt to show that optimum solutions can be arrived at by the computer; thus the main question of creative design versus computer design has to be discussed.

Dicke[2] deals with the optimum combination of safety and economy which is actually the most important point in connection with decision making in the creative design of a structure for which the criteria of safety and reliability are of the greatest importance. "Fire Effects" which comes under Theme (III) have also been briefly included to cover the overall aspect of safety.

The first writer already dealt with collapse load design as early as 1933 as a Consulting Engineer in Vienna in his capacity as advisor to a spun concrete works regarding the design of centrifugally moulded high strength concrete poles. These had to be designed for a definite load factor against collapse (based on the worst case of combined wind, snow and ice so far established), according to the design regulation of the Association of Electrical Engineers. Thus he had ample opportunities to deal with the problems of safety and ultimate load design before this subject became the topic of general discussion. In his further activity with British Railways Eastern Region he had the opportunity in 1948-1962 of introducing, in spite of great opposition, "partial prestressing" for bridges and structures on a safe and economical basis[3].

The second author, as Senior Partner of a firm of Consulting Engineers, has dealt with the question of creative design in some publications.[4,5,6]. The three authors have discussed the specific design problems of controlling the dynamic effect of wind on light cantilever roofs by external damping.[7]

In this paper the safety requirements are particularly emphasised, and the question of creative design is discussed on the basis of reference to some of the outstanding pioneers of reinforced and prestressed concrete design.

2. The Required Factor of Safety and Limit State Design

The first author had discussed this question in 1968[8]. Consider five simple principles from the book "Introduction to Prestressed Concrete".[9]

1) There is no progress without considered risk.
2) The proof of the pudding is in the eating.
3) We live and learn.
4) Do not generalise, rather qualify the specific circumstances.
5) It does not matter how cheap but how good a thing is.

With no progress we would still live, as our forebears did, in the Stone Age, Principles 1 and 2 can be combined, as preliminary tests may prove the suitability of new developments. "Considered risk" obviously does not mean gambling, as it must be based on preliminary studies according to principle 2. Nevertheless, it is possible that some properties of new structures or materials become known only at a later stage, which leads to principle 3. Obviously it is a question of economy to select a solution which is safe for definite conditions although not absolutely safe (which is impossible). However, it could be dangerous and false economy to select a solution which is cheap but not good enough (principle 5). The designer ought to be warned against optimisation for optimisation's sake, since the consequences of failure must always be borne in mind.

The question of the "desirable factor of safety" has been closely investigated by Freudenthal in many papers since 1945, as enumerated in [8] and also in the IABSE Congresses in New York (1968) and Amsterdam (1972). Already in the first paper in 1945 the importance of probabilistic methods and statistics of loading were stressed as an integral part of rational design. The concept of safety was developed as a problem of uncertain predictions of the performance of structural materials as well as of the magnitude of the load pattern. Also the need for serviceability was stressed; Freudenthal stated: "The safety factor is thus transformed into a parameter that is a function of the random variation of all design characteristics as well as of the non-random variations, essentially caused by the process of construction". To this, in the authors' opinion, should be added the parameter covering the consequences of failure and the practical implications of the non-random variations mentioned above, as discussed later.

Paez and Torroja[10] were the first to publish a book on this problem in 1950 and referred to the importance of economical considerations, indicating that the question of insurance premium and the cost of construction and probable indemnity for possible losses should be considered. This leads to the question of "calculated risk" which traditionally applied to a great extent to earthworks and foundation engineering, but which relates also to earthquakes, the effects of explosions and even to excessive high wind pressure and other accidental overloading. Casagrande[11] considers "two distinct steps" for the definition of "calculated risk": "(a) the use of imperfect knowledge, guided by judgement and experience, to estimate the probable ranges for all pertinent quantities that enter into the solution of a problem, and (b) the decision on an appropriate margin of safety, or degree of risk, taking into consideration economic factors and the magnitude of losses that would result from failure." This shows that the question of probability when extended to include unknown quantities becomes very involved.

The classical theory of probability, based solely on a statistical study of load and strength, would lead to the same safety factors for all elements of a structure regardless of their importance. The subjective or Bayesian theory is based on individual jüdgement. The decision making in this case is a part of the problem, including the admission of subjective probabilities. This has been partly embodied in the modern Codes by partial safety factors. However, many features still remain shrouded in the mysteries of engineering judgement. This question will be discussed more closely later. The problems of safety have been discussed for the last 10 years in Committee 348 "Structural Safety" of the American Concrete Institute, of which the first author is a member. One point where subjective considerations may apply is the selection of different factors of safety for specific structural members, e.g. a column carrying many storeys should have a greater margin in safety than a beam of minor importance. Although the probability of simultaneous overloading of all floors is very small, the cost of increased carrying capacity of a column is insignificant as compared with the entire cost of a building. Collapse of buildings has mostly been caused by an accumulation of mistakes in design and/or construction details, bad materials and unsatisfactory workmanship. Nevertheless, there always appears to be a dominant factor.

There is obviously no absolute safety of structures; e.g. an aircraft disaster may occur and an aeroplane may fall on houses causing destruction, or an atomic attack of unforeseen magnitude may occur. Obviously structures cannot be built to resist such possible, but improbable, events. However, it is suggested to design nuclear reactors strong enough to resist the dropping down of an aircraft, based on certain loads and impact.[12] The likelihood that earthquakes might occur is limited to districts where, based on experience, such catastrophies may occur and where the degree is classified by former events. Similarly, districts where special winds of high turbulence may occur (such as hurricanes and tornadoes) have to be dealt with separately. If these extraordinary conditions are excluded the following conditions ought to be taken into account:

a) Serviceability under working load, including fire resistance for a pre-determined time and the effect of maximum possible temperature changes due to environmental conditions.

b) Safety against collapse for ultimate limit conditions including fire resistance for a pre-determined time.

c) Capacity to absorb impact in the event of shock (such as an explosion).

d) Resistance to weathering including freezing if the structure is in the open for the expected life time.

In respect of working load, the effect of fatigue and vibration due to wind turbulence (particularly with light roofs) also needs to be taken into account. The importance of dealing with this problem has been discussed in the papers [7&13]. When excitation due to vortex oscillation applies, there are three basic possibilities: (i) continuous oscillation with varying amplitude which might cumulatively lead to fatigue failure; (ii) instability (flutter) both with high and low wind velocity; and (iii) resonance. The behaviour of each structure should therefore, where necessary, be aero-dynamically examined in a model test in a wind tunnel for various wind speeds and it may be desirable to provide external damping devices to reduce the amplitude of vibration, particularly of light roof structures. Davenport[14] stated in 1975 that the question of damping will be the most important problem to be investigated until the Fifth International Conference on Wind Effects.

3. The Fire Resistance of Reinforced and Prestressed Concrete

Kawagoe and Saito[15] state that "reinforced or prestressed concrete structures have explosively spalled in the early stage of fire". In the Report on "The Fire Resistance of Concrete Structures"[16] by a Committee of which the second author was Chairman, this question has been extensively dealt with. It depends greatly on the aggregates; calcareous materials are much less likely to spall than siliceaous aggregates such as flint gravel, granite and crushed stone. Three types of spalling are distinguished: (a) destructive spalling; (b) local spalling; and (c) sloughing off, which is a gradual progressive form of breakdown that may continue slowly through the later stages of heating. At many fire tests and actual fires spalling was not noticed, particularly with lightweight concrete made with sintered pulverised fuel ash aggregates. Obviously "explosive" spalling ought to be avoided by particular specification and selection of the aggregates, or detailing, or by a combination of both. Suitable detailing can take into account even considerable amounts of spalling without critically endangering the load carrying capacity[17].

Restraint of the member is of particular importance and a very favourable behaviour can be obtained as the authors have shown in the paper[18] where the deflection after 1½ hours fire was very small, although the temperature in the steel was very high. Based on the assumption that spalling can be minimised the construction can be designed to resist a required time and still remain serviceable and to avoid collapse for another required time, which would correspond to the conditions (a) and (b) described before.

Gustaferro[19] deals in his paper with a rational design for fire resistance. He appears to be of the opinion that spalling is not critical in the majority of cases, especially with well detailed constructions. The authors have in the second edition of "Prestressed Concrete Designer's Handbook"[20] shown that it is possible, and advisable, to design a member to resist fire for a definite time. This is again also a question of satisfactory detailing.

4. Elastic and Plastic Behaviour and its role in the Development of Reinforced and Prestressed Concrete

In the last quarter of the nineteenth century, the basic behaviour of the co-operation between concrete and steel was recognised. At the beginning of this century relatively highly-developed reinforced concrete structures were built by Hennebique, the creator of the T-beam, in France, Mensch in USA, Emperger in Austria, Maillart in Switzerland and Danusso in Italy.

These pioneers based their design computations directly on some large scale failure tests, without much reference to the actual distribution of stresses. Their method based on experience ensured already a definite load factor of safety against failure, combined with satisfactory behaviour under working load. An example of this kind of creation is Hennebique's famous bridge over the Tiber River in Rome, built in 1912, which is so slender and elegant that it would hardly be possible to replace it by a more slender solution today. Another outstanding example is the bridge at Liége in Belgium also designed by him.

The design of reinforced concrete was investigated by a French Commission in 1906 and their report (which was based on the elastic theory proposed by the German Professor Mörsch) recommended that the tensile strength of concrete be ignored, when considering resistance to bending. This "elastic" theory was accepted as the fundamental basis for reinforced concrete design until the more recent general acceptance of the ultimate load theory.

Olsszak presented at the New York IABSE Congress[21] an ingenious contribution to the question "elasticity" and "plasticity". He said "A reversible (or elastic) deformation, as you all know, is the response of a material in the first stage of loading process." "A plastic deformation is a kind of defence (self defence) of the material against overloading." and "A conscientious designer wants to know what really is going to happen to his structure in the course of its existence, let us say, in a year, or two, or five; or perhaps what is going to happen if the structure - by accident or purpose - is overloaded." Thus there is no contradiction, and of course no competition between "elastic" and "plastic" approaches.

Nervi[22] states that plasticity could be used to improve the re-distribution of stresses and makes special reference to the ingenious intuitive design of Hennebique's bridges as mentioned before. Nervi, himself a builder, combined the function of master builder and architect, and his designs were based on intuition and model tests. By the use of the very suitable micro-concrete he was able to minimise the use of materials and to employ reliable, first class craftsmen, thus ensuring the high quality of his structures combined with economy.

The creative design based on intuition, imagination and knowledge which became apparent in the work of the first pioneers of reinforced concrete and Nervi, as discussed above, will be dealt with in connection with describing the philosophy of Eduardo Torroja. The views of Freyssinet, the creator of the "elastic" design of prestressed concrete, should also be noted. Freyssinet stipulated that the structural member should be under "permanent" compression and as late as 1950 was of the opinion that any "half-way house" between reinforced and prestressed concrete was bad.[23] This is very surprising, bearing in mind his outstanding reputation as a designer of bridges and other structures based on his intuition. Thus, "permanent nominal compression" was a "must" for the prestressed concrete designer for some time. This claim is still considered essential by some authorities even today. However, within the FIP, discussion is proceeding with a view to the acceptance of limited cracking under service load, provided that there is no danger of corrosion. In fact, it has been suggested that reinforced concrete and fully prestressed concrete are the extreme conditions, with partially prestressed concrete as the general case. As in life, so in design, the middle of the road solution may be more satisfactory than the extremes, as the authors stated in the preface of [20].

5. Creative Design

Eduardo Torroja has dealt with this problem in a unique manner in his books [24] and [25]. He states: "The calculation of stresses can only serve to check and to correct the sizes of the structural members as conceived and proposed by the intuition of the designer." "For the sake of both stability and strength the first essential task is to determine accurately all possible loads and effects to which a structure is submitted." "The designer must obviously have full knowledge of the properties of materials and behaviour of structures, but when designing a structure of any kind, the ultimate purpose of the building has to be studied from every angle. This may be described by his design of the famous shell construction over the Madrid Racecourse in 1935. He came to the conclusion that it was essential as a safeguard against wind forces that the main support be rigidly fixed at the promenade level, the roof having a good stability by the general arrangement provided of having a massive column at the cantilever support and a vertical steel tie instead of a column at the other support. Several possible solutions were investigated, such as a conoid and hyperboloid. Torroja states in[25]: "Is the invention of a specially adapted form to solve a specific problem strictly an imaginative process, or is it the result of logical reasoning based on logical training. I do not think it is either of the two but rather both together. The imagination alone could not have produced such a design unaided by reason, nor could a process of deduction, advancing by successive cycles of refinement, have been so logical and determinate as to lead inevitably to it - whatever the reader of these lines may have inferred."

This is a perfect analysis of the problem of creative design. Obviously in addition to knowledge also technical experience must be available. In fact, knowledge is mainly obtained by what we have learned from our own shortcomings or from those of others. Another example of the works by Torroja may be mentioned; his design of the "Tempul Aqueduct" in 1925, at the age of 26 years. He was the first to apply prestressing high tensile cables in order to be able to omit two supports in difficult foundations in the river, and thus created the first prestressed suspended bridge. Also his process of obtaining prestressing shows the combination of simplicity and efficiency.

Torroja says in the preface that his "final aim has always been for the functional, structural and aesthetic aspect of a project to present an integrated whole both in essence and appearance". Unfortunately there are few, if any, designers able to follow Torroja's ingenuity, but it should be aimed at.

6. Safety-Optimisation

The question of economy leads often to investigations of optimsation in order to obtain the minimum of materials possible. This is quite often not the cheapest solution, as the general conditions of labour and transport have also to be considered. Nevertheless, the question of rationalisation needs to be discussed. Thompson and Hunt[26] have discussed the dangers which may occur by an increasing degree of optimisation "an increasingly unstable failure characteristic" may result, which ought to be well considered by civil and structural engineers, but need not perhaps be fully taken into account by a "weight conscious aircraft designer" who intends to "seek the highest possible optimisation allowing the best he can for the random manufacturing tolerances."

Thompson[27] deals in his paper with the new, so-called "catastrophe" theories (by R. Thom and Zeeman) which predict that "buckling strength can be dramatically eroded by small unavoidable manufacturing imperfections". Chilver[28] warns against the search for the lightest and therefore most efficient structural forms which may lead to potentially catastrophic engineering structures, although there are pressures put on the designer to converge on an optimal solution.

These considerations which mainly apply to buckling and stability problems ought to be applied generally when considering the partial safety factor for the material. Factory made precast concrete ought to be permitted lower values of partial safety factor for materials only when the designer is fully satisfied that these will be made under the supervision of a reliable, experienced and qualified engineer. Often this is not the case, even in factories, whereas in fact it can be achieved at a well organised building site.

Figure 1 lists the basic stages of supervision and explains the principles and the assumptions under which they are applied. By underlining certain parts of the explanatory notes the interdependence of the basic control method is stressed. Figure 2 gives a qualitative illustration of the interaction of production and independent supervision for various materials. While site standards of supervision give a relatively small drop along curve A, it results in a catastrophic drop along curve B.

In addition to the partial safety factor covering materials γ_m and another covering loads γ_f, it is desirable to introduce a third partial factor γ_c, covering the nature of the structure or member and the consequences of failure[29]. This latter factor can be considered in two stages (γ_{c1} and γ_{c2}). Firstly the nature of the structure and its behaviour, e.g. brittle or ductile, series or parallel assembly of members, is accounted for. Secondly, the seriousness of failure in human and economic terms is allowed for. Such criteria in fact renders explicit what has always been implicit in the thinking of the designer and permits us to clarify, indeed enumerate, an aspect of design hitherto hidden in the mysteries of engineering judgement.

7. Conclusions

The combination of a creative and economical design may be briefly outlined by statements of the second author, taken from his papers[4, 5 & 30]. He said that the ultimate achievement in creative design can only be truly measured by the closeness with which one approaches the unobtainable. Evolutionary processes in nature ensure that only the most rational forms pass the real test of survival.

The eagerness to calculate, rather than to think, coupled with a traditional 'Bill-of-Quantities' mentality is the main reason why many designs are unsatisfactory. Designers often are ignorant of production methods and often basic design decisions are taken by bodies independently of the designer. It is commonplace for the contractor, through inadequate pricing, to virtually subsidise bad designs, while good and progressive designs are often priced out of the market. "Design without imagination is a contradiction in itself. However, the designer with imagination but with insufficient knowledge of technology of the medium in which he designs, is not only wasteful, but indeed dangerous."[30]

In principle there are three types of control:

INITIAL TESTS

STATISTICAL ESTIMATION OF THE PROPERTIES OF THE MATERIAL TO BE APPLIED

CONTINUOUS PRODUCTION CONTROL

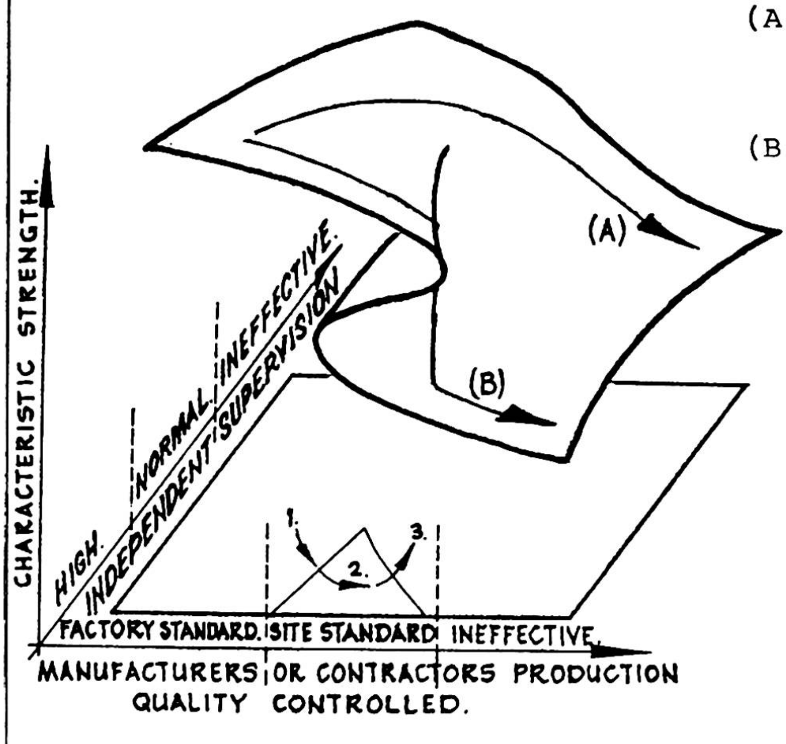
TO ENSURE STABILITY OF THE PRODUCTION PROCESS AND FOR THE CONTROL OF THE PRODUCTION (FACTORY CONTROL)

COMPLIANCE CONTROL

JUDGEMENT OF ACCEPTABILITY OF QUALITY

Although the three types of control are the responsibility of different persons involved in the process of the realisation of a structure, they serve, together with the design, in achieving the construction of economic and safe buildings. This achievement can, however, only be reached by a clear distribution of the responsibilities. As a rule, control systems have the greatest efficiency, if the producer accepts responsibility for the costs of control as well as the consequences of negative decisions.
It should be emphasised that the rules for the dimensioning, the type and extent of initial tests, the expense for the production control and the strictness of the compliance control interact with each other.

FIGURE 1 Extract from CEB Bulletin D'Information 111, October 1975



(A) Portland cement concrete (traditional way); mild steel.

(B) High strength concrete and concrete made with high alumina cement or with calcium chloride; high tensile steel; armoured glass; laminated timber; notch sensitive plastic; (all requiring care)

FIGURE 2 Qualitative illustration of the influence on characteristic strength of materials, of interaction of production control and independent supervision, using catastrophe presentation according to ref. 27.

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SUMMARY

Safety of structures for normal and abnormal loading (including fire) are discussed. "Calculated risk" is enumerated by three basic partial factors of safety: material (γ_m) Loading (γ_f) and mode of failure with consequences (γ_c). Creativity, a pre-requisite of conceptual design, balancing safety and economy, cannot be expected from the computer which serves only as a tool.

RESUME

La sécurité des structures est présentée pour des cas de charge normaux et anormaux (incendie inclus). Le "risque calculé" dépend des trois coefficients partiels de sécurité: Matériau (γ_m), charge (γ_f), genre de rupture et conséquences (γ_c). La créativité - condition essentielle d'une conception basée sur la sécurité et l'économie - ne peut pas provenir de l'ordinateur, qui n'est qu'un instrument de calcul.

ZUSAMMENFASSUNG

Die Sicherheit von Tragwerken für die gewöhnliche und aussergewöhnliche Belastung (inkl. Feuersicherheit) wird diskutiert. "Kalkuliertes Risiko" besteht aus drei elementaren Sicherheitsfaktoren: Material (γ_m), Belastung (γ_f) und Bruchart mit Folgen (γ_c). Kreativität, als Zusammenspiel eines zwischen Sicherheit und Wirtschaftlichkeit ausgewogenen Entwurfs kann vom Computer nicht erwartet werden. Dieser dient nur als Hilfsmittel.

Sensitivité de la sécurité des constructions par rapport aux types de comportements structuraux

Einfluss des Tragverhaltens auf die Tragwerksicherheit

Sensitivity of Structural Reliability to different Types of Structural Behaviour

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1. INTRODUCTION.

L'idée de base du rapport introductif de DICKE [2], est de promouvoir les méthodes de calcul basées sur le concept probabiliste de la sécurité, qui soient accessibles à l'ingénieur praticien. Les coefficients de sécurité utilisés dans de telles méthodes doivent tenir compte de la dispersion inhérente aux actions et aux caractéristiques structurales, mais aussi du type de comportement de la structure.

L'objet de cette étude est d'examiner, du point de vue probabiliste, deux comportements structuraux particuliers, à savoir celui du type "chaîne" et celui du type ductile. En s'inspirant des travaux de MOSES et TICHY [10], [11], de ROSENBLUETH [12] et des auteurs [3], [5 à 7], on propose, pour chacun de ces deux types de comportement, des modèles probabilistes permettant de déterminer les valeurs des coefficients reliant la sécurité d'une section (ou d'un élément) à la sécurité d'ensemble de la structure. On se propose aussi d'examiner la sensitivité de ces coefficients vis-à-vis de différents paramètres qui ne peuvent pas être considérés par une approche classique déterministe.

On arrive ainsi à mettre à la portée des commissions de rédaction de codes un ensemble de résultats, qui peuvent constituer une base de départ pour l'établissement des valeurs définitives des coefficients envisagés.

2. COMPORTEMENT DU TYPE "CHAINE".

Ce type de comportement est caractéristique des constructions dont la ruine d'un élément (ou d'une section) amène la ruine d'ensemble de la structure. Ceci est valable pour le dimensionnement à la ruine des structures isostatiques, puisqu'il n'y a pas de possibilité d'adaptation plastique entre sections. On peut aussi y assimiler le dimensionnement élastique puisque, dans ce cas, on considère que la structure est mise hors service lorsque l'on atteint la sollicitation limite dans une section. Les constructions en grands panneaux sont des exemples de structures présentant un comportement du type "chaîne".

Soit N le nombre d'éléments (sections critiques) d'une telle structure. Appelons C_i et S_i les variables aléatoires représentant respectivement la capacité portante et l'effet des actions dans l'élément i. Si $R_i = C_i - S_i$ désigne la réserve de sécurité de l'élément en cause, et si E_i est l'événement $R_i = C_i - S_i > 0$, la probabilité d'apparition de l'événement

$$E = E_1 \cap E_2 \cap \ldots \cap E_i \cap \ldots \cap E_N$$

représente la probabilité de survie $P_{(+)}$ qui mesure le degré de sécurité de la structure.

Le calcul de cette probabilité est pratiquement impossible [8]. Néanmoins, il existe des modèles permettant de trouver les bornes d'un intervalle qui encadre sa valeur exacte [1] , [3] :

$$\prod_{i=i}^{N} P_{(+)i} \leqslant P_{(+)} \leqslant \min (P_{(+)i}) \qquad (3)$$

Ces bornes sont beaucoup plus faciles à calculer.

La borne inférieure s'obtient en faisant l'hypothèse que les réserves de sécurité R_i des éléments sont statistiquement indépendantes. Dans ce cas, les coefficients de corrélation entre les réserves de sécurité sont tous nuls:

$$\rho(R_i, R_j) \equiv 0 \text{ si } i \neq j.$$

La borne supérieure s'obtient en supposant qu'il existe une corrélation positive parfaite entre les réserves de sécurité R_i des éléments. Dans ce cas, les coefficients de corrélation entre les réserves de sécurité sont tous égaux à l'unité :

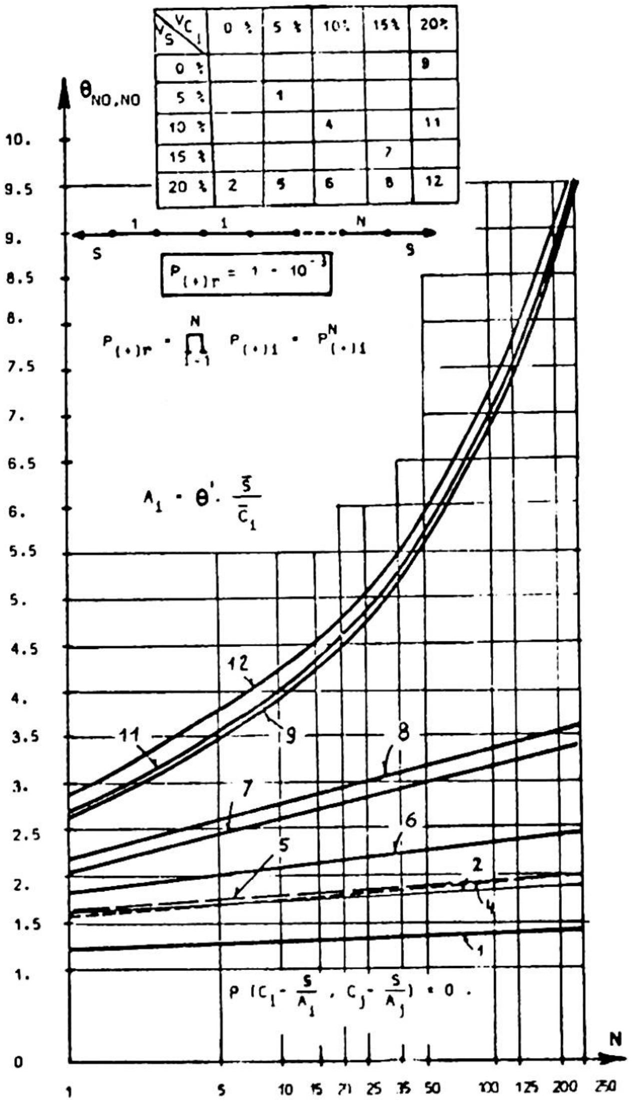
$$\rho(R_i , R_j) \equiv 1$$

Puisque le degré de dépendance corrélative entre les variables n'est généralement pas connu, il est recommandable de se placer du côté de la sécurité et d'utiliser la borne inférieure.

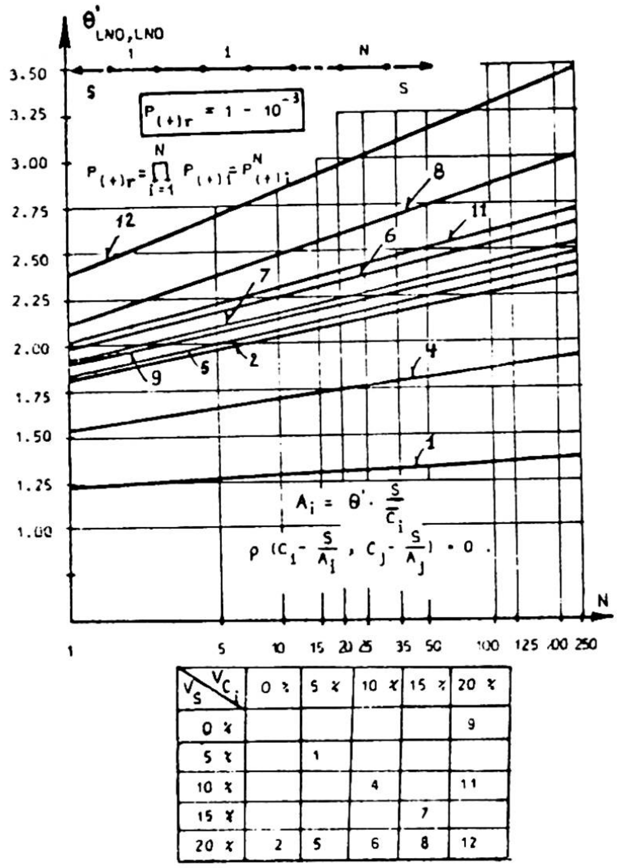
En général, on impose que la probabilité de survie de la structure soit au moins égale à une valeur de référence $P_{(+)r}$, acceptée à priori. Le problème consiste alors à trouver, à partir de cette sécurité d'ensemble de la structure, un modèle donnant la sécurité de chacun des éléments, et permettant ainsi leur dimensionnement.

On utilise dans ce but le modèle classique d'une chaîne, constituée de N maillons identiques, de section A_i (supposée non aléatoire), soumise à traction par l'action aléatoire S [4], [11], [12]. En identifiant $P_{(+)}$ à la borne inférieure définie en (3), donc en se plaçant du côté de la sécurité , il résulte que chaque élément (schématisé par un maillon) doit avoir une probabilité de survie égale à $\sqrt[N]{P_{(+)r}}$.

A la figure 1 [5], on a représenté la variation, en fonction du nombre d'éléments N, du coefficient central de sécurité θ nécessaire à chaque élément pour assurer une probabilité de survie $P_{(+)r} = 1 - 10^{-3}$ de la structure.



- FIG. 1 -



- FIG. 2 -

On a effectué diverses combinaisons des coefficients de variation de l'action V_S et des capacités portantes des éléments V_{C_i}. Les variables S et C_i sont supposées avoir une loi de distribution normale, si bien que $\theta = \theta'_{NO,NO}$.

La figure 2 [5] donne la variation du même coefficient, mais dans le cas où les variables ont une distribution logarithmique-normale. On remarque cette fois que le coefficient central de sécurité $\theta'_{LNO,LNO}$ varie linéairement avec le logarithme népérien du nombre d'éléments (N est indiqué en abscisse à l'échelle logarithmique).

L'examen des figures précédentes permet de mettre en évidence la sensitivité du coefficient central de sécurité θ vis-à-vis des différents paramètres considérés. On remarque que ce coefficient augmente avec le nombre d'éléments et avec la dispersion de l'action et des capacités portantes. Il est aussi influencé par le type de distribution (comparer les figures 1 et 2); il augmente avec la probabilité de survie à assurer à la structure.

3. COMPORTEMENT DU TYPE DUCTILE.

Ce type de comportement est caractéristique des structures en acier doux ou en béton faiblement armé, dans lesquelles les sections présentent une ductilité suffisante pour qu'une adaptation plastique entre sections puisse avoir lieu.

La sécurité d'une telle structure vis-à-vis d'un mode de ruine particulier dépend de la capacité portante des N sections critiques associées à ce mode. Le modèle qui permet de déterminer la sécurité des sections critiques, en fonction de la probabilité de survie de l'ensemble de la structure, est formé de N barres parallèles identiques, de section A_i (supposée non aléatoire), soumises à traction par l'action aléatoire S.

La probabilité de survie de ce système s'écrit :

$$P_{(+)} = P\left(\sum_{i=1}^{N} A_i C_i > S\right) \tag{4}$$

où $A_i C_i$ représente la capacité portante de la i-ème barre. Si toutes les capacités portantes des barres sont des variables aléatoires indépendantes caractérisées par la même espérance mathématique et la même variance, le coefficient de variation de la capacité portante du système vaut [11], [9] :

$$V_{\sum_{i=1}^{N} A_i C_i} = \frac{V_{C_i}}{\sqrt{N}} \tag{5}$$

Ce coefficient est donc $\sqrt{N}$ fois plus petit que le coefficient de variation de la capacité portante d'une barre.

Dans ces conditions, la figure 3 [5] représente la variation, en fonction du nombre de sections critiques N, du coefficient central de sécurité, $\theta''_{NO,NO}$, nécessaire à chaque section critique (schématisée par une barre) pour assurer la probabilité de survie $P_{(+)r} = 1 - 10^{-3}$ de la structure (schématisée par l'ensemble des barres). On a effectué diverses combinaisons des coefficients de variation de l'action V_S et des capacités portantes des sections critiques V_{C_i}. Les variables S et C_i sont supposées avoir une loi de distribution normale.
Dans le cas où ces variables obéissent à une loi logarithmique-normale, le coefficient central de sécurité $\theta''_{LNO,LNO}$ est donné à la figure 4 [5] pour une probabilité de survie identique.

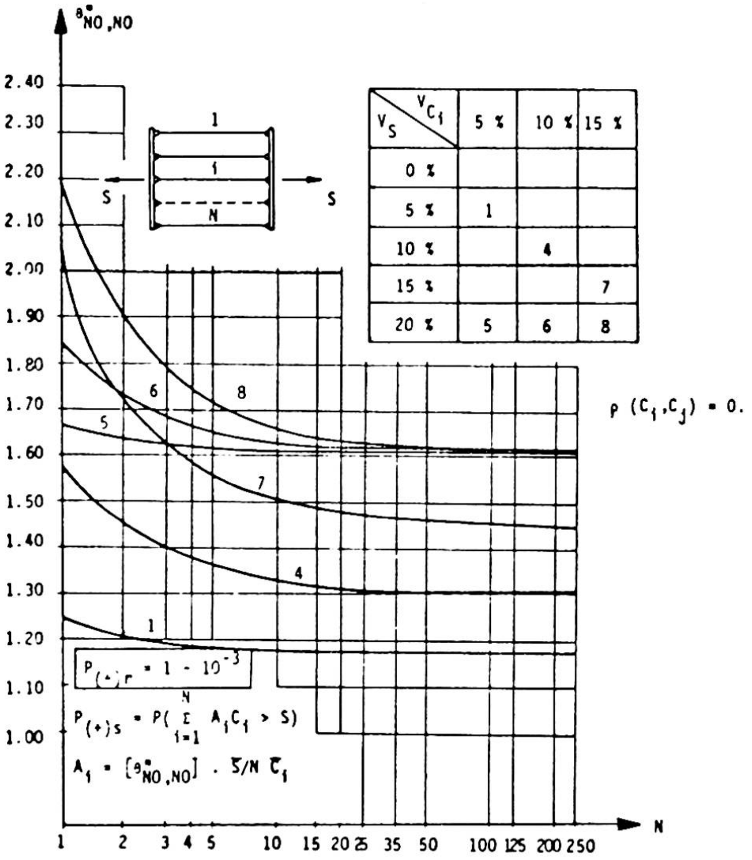
L'analyse des figures 3 et 4 montre que la diminution du coefficient θ'' est surtout accentuée dans le domaine des faibles valeurs de N ($N < 5$).
Il faut aussi noter que, lorsque N devient important ($N > 10$), ces coefficients sont nettement plus sensibles, pour une même valeur de N, à une variation de V_S que de V_{C_i}.

4. CONCLUSIONS.

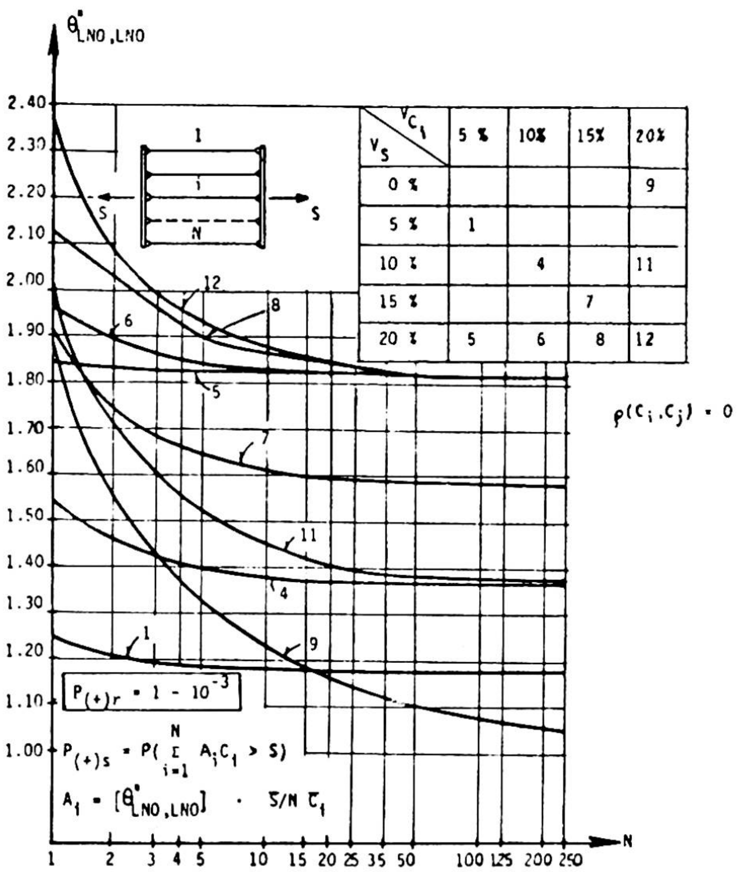
Cette étude nous permet de tirer les conclusions suivantes :

1. La sécurité d'un élément qui fait partie d'une structure présentant un comportement du type "chaîne" est toujours supérieure à la sécurité d'ensemble de la structure. Le danger de ruine est maximum quand les réserves de sécurité des éléments qui composent la structure sont des variables aléatoires indépendantes.

Le coefficient θ' qui est une fonction croissante de N, V_S, V_{C_i} et $P_{(+)r}$



- FIG. 3 -



- FIG. 4 -

permet de trouver la sécurité d'un élément en fonction de la sécurité d'ensemble de la structure. Ce coefficient conduit parfois à une estimation trop pessimiste du danger de ruine; c'est le cas lorsqu'il existe une corrélation positive importante entre les capacités portantes des éléments.

2. Dans le cas d'une structure présentant un comportement du type ductile, la sécurité d'une section critique associée à un mécanisme de ruine est toujours inférieure à la sécurité de la structure vis-à-vis de l'apparition du mécanisme considéré. La probabilité d'occurrence de ce mécanisme est minimum quand les capacités portantes des sections critiques sont statistiquement indépendantes. Le coefficient θ", qui est une fonction croissante de V_S, V_{C_i}, $P_{(+)r}$, diminue lorsque le nombre N de sections critiques augmente. Ce coefficient est parfois trop optimiste; c'est le cas lorsqu'il existe une corrélation positive importante entre les capacités portantes des sections critiques.

En raison de la complexité des concepts théoriques et des calculs qui aboutissent à exprimer la sécurité au niveau d'une section en fonction de la sécurité d'ensemble de la structure, les raisonnements ont été effectués sur des modèles très schématiques. C'est pourquoi on ne doit pas conclure que les coefficients θ' et θ" présentés aux diagrammes précédents constituent des valeurs directement utilisables dans la pratique. Dans l'esprit des auteurs, elles doivent plutôt servir comme base de référence pour les commissions chargées de l'élaboration des codes de construction.

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RESUME

On utilise des modèles probabilistes pour analyser l'influence de divers paramètres sur la sécurité des structures présentant un comportement de type "chaîne" ou de type ductile. On aboutit à des coefficients probabilistes qui relient la sécurité d'un élément à la sécurité d'ensemble de la structure. Les résultats montrent leur sensitivité vis-à-vis des différents paramètres.

ZUSAMMENFASSUNG

Anhand von Wahrscheinlichkeitsmodellen wird der Einfluss verschiedener Parameter auf die Sicherheit von Tragwerken, welche dem "Kettentyp" oder dem duktilen Typ entsprechen, untersucht. Man findet Koeffizienten, welche den Zusammenhang der Sicherheit des Einzelteils zur Systemsicherheit ausdrücken. Die Ergebnisse zeigen ihre Empfindlichkeit auf die verschiedenen Parameter.

SUMMARY

Probabilistic models are used for the analysis of the influence of various parameters on the reliability of structures presenting a "weakest-link" or ductile type behaviour. Both cases lead to probabilistic coefficients which link the element reliability to the structure reliability. The results show their sensitivity to the different parameters.

Ib

Sicherheit und Wirtschaftlichkeit von Bauwerken mit grosser Anwendungsbreite

Safety and Economy of Buildings with a Wide Range of Applications

Sécurité et économie des bâtiments à l'usage universel

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In seinem interessanten Vorbericht zum Thema Ib umreißt Prof. Dr. Dicke, ausgehend von realen Situationen und praktischen Vorstellungen, die aktuellen Probleme der Relation von Sicherheit, Zuverlässigkeit und Ökonomie. Die Tatsache, daß zwischen der schnellen Entwicklung der Zuverlässigkeitstheorie und ihrer praktischen Nutzung im Bauwesen eine Kluft entstanden ist, erfordert einfache Regeln für die Anwendung der bisher vorliegenden Erkenntnisse, wobei auch Kompromisse in Kauf zu nehmen sind.

Wir alle wissen um die Vielfalt und Komplexität der Faktoren, die die Sicherheit und Zuverlässigkeit der Bauwerke beeinflussen. Sie schließen u.a. die Homogenität der Materialstruktur, die Wirkungsweise des statischen Systems, die Belastungen unterschiedlichster Art, die Auswirkungen der Fertigungs- und Baustellenprozesse, der Korrosion, von Explosionen und Bränden ein. Es muß deshalb vermieden werden, daß die auf zuverlässigkeitstheoretischen Erkenntnissen aufbauenden verfeinerten Berechnungsmethoden, trotz des Einsatzes der elektronischen Rechentechnik, zu einem unvertretbaren Anschwellen des Projektierungsaufwandes führen.

Ein praktikabler Weg zur Umsetzung der probabilistischen Sicherheitsphilosophie in die Praxis besteht in der Herausarbeitung bindender Bedingungen für einfache Näherungen an die neuen Sicherheitskonzeptionen. Am wirkungsvollsten ist das für solche Bauwerke zu erreichen, die in großen Serien aus industriell vorgefertigten Elementen in der Montagebauweise errichtet werden. Bei solchen industriellen Bauweisen sind die aus den verfeinerten Berechnungsverfahren und der höheren Ausnutzung der Baustoffeigenschaften sich ergebenden Anforderungen an die Qualität der Elementefertigung und Bauausführung am besten zu gewährleisten.

In der Deutschen Demokratischen Republik wird der Entwicklung des industriellen Bauens große Aufmerksamkeit gewidmet. So werden z.B. jährlich ca. 1oo.ooo Wohnungen in der Großplattenbauweise errichtet; in großem Umfang kommen vorgefertigte Skelettkonstruktionen aus Stahlbeton und als Metalleichtbauten für ein- und mehrgeschossige Gebäude der Industrie, für gesellschaftliche Einrichtungen und für die Landwirtschaft zur Anwendung. Dazu wurden von der Bauakademie der DDR, als der zentralen Forschungseinrichtung des Bauwesens in unserem Lande, umfangreiche Forschungs- und Entwicklungsarbeiten zu einheitlichen funktionellen, konstruktiven und technologischen Lösungen in enger Zusammenarbeit mit Projektierungseinrichtungen und bauausführenden Betrieben durchgeführt. In unsere Betrachtungen werden ebenfalls die Bedingungen der Nutzung und Instandhaltung unter Berücksichtigung der unterschiedlichen moralischen und physischen Verschleißzyklen der Trag- und Ausbaukonstruktionen einbezogen. Damit werden Voraussetzungen für eine planmäßig vorbeugende Instandhaltung und einen optimalen Materialeinsatz in Relation zum Nutzungszeitraum und zum Verschleiß geschaffen. Fragen der Zuverlässigkeit und Sicherheit der Baukonstruktionen und Bauwerke spielen dabei durchweg eine bedeutende Rolle. Über einige Arbeiten dieser Art wird im folgenden berichtet.

1. Berechnung von Versagenswahrscheinlichkeiten und praktische Schlußfolgerungen

In den vergangenen Jahren wurden grundlegende Untersuchungen zur theoretischen Beurteilung und quantitativen Bestimmung von Sicherheitsfestlegungen in Berechnungsvorschriften durchgeführt, wobei die Wechselwirkung zwischen Überlebens- bzw. Versagenswahrscheinlichkeit und Materialaufwand eine besondere Rolle spielte [1].

Methoden zur Berechnung der Versagenswahrscheinlichkeit sind zwar bekannt. Wegen der Kompliziertheit solcher Verfahren und des offensichtlichen Mangels an den erforderlichen statistischen Daten scheidet eine direkte Bemessung nach der zulässigen Versagenswahrscheinlichkeit in absehbarer Zeit aus. Will man zu grundlegenden Folgerungen kommen, interessiert auch nicht so sehr die Versagenswahrscheinlichkeit eines einzelnen Tragwerks, sondern das Niveau und die Veränderlichkeit der Versagenswahrscheinlichkeit für eine ganze Tragwerksklasse, die nach gleichen Sicherheitsfestlegungen bemessen wird.

Nach diesem Prinzip wurden stählerne Dachtragewerke für den Versagensfall Stahlfließen unter Schneelast untersucht [2]. Sie sind repräsentativ für die Masse der serienmäßig ausgeführten leichten Dächer, bei denen neben der Eigenlast die Schneelast den Haupteinfluß auf die Zuverlässigkeit ausübt. Zunächst wurden umfangreiche statistische Daten über die Schneelast, die Eigenlast, den Baustoff Stahl und einige wichtige Parameter der zu untersuchenden Tragwerksklasse erfaßt und aufbereitet. Schon erste Untersuchungen zeigten, daß nach geltenden Vorschriften bemessene Dachtragwerke keineswegs die gleiche Versagenswahrscheinlichkeit haben. In Abhängigkeit von der Höhenlage ü. NN und von der Größe der Eigenlast des Daches schwankt diese sehr stark. Das leichte Dach im Gebirge hat z.B. eine um mehrere Zehnerpotenzen höhere Versagenswahrscheinlichkeit als ein schweres Dach im Flachland.

Zur systematischen Analyse von verschiedenen für den vorliegenden Fall gültigen Vorschriftensystemen wurde ein geeignetes mathematisches Modell entwickelt und auf insgesamt 11 Vorschriftensysteme angewendet. Es läßt sich damit die zweidimensionale Häufigkeitsverteilung von Versagenswahrscheinlichkeit und Materialaufwand berechnen und darstellen. Das Modell arbeitet nach der Monte Carlo Methode, die Ergebnisse werden unter Nutzung der EDV von einem Zeichenautomaten ausgewertet (vergleiche Bild 1).

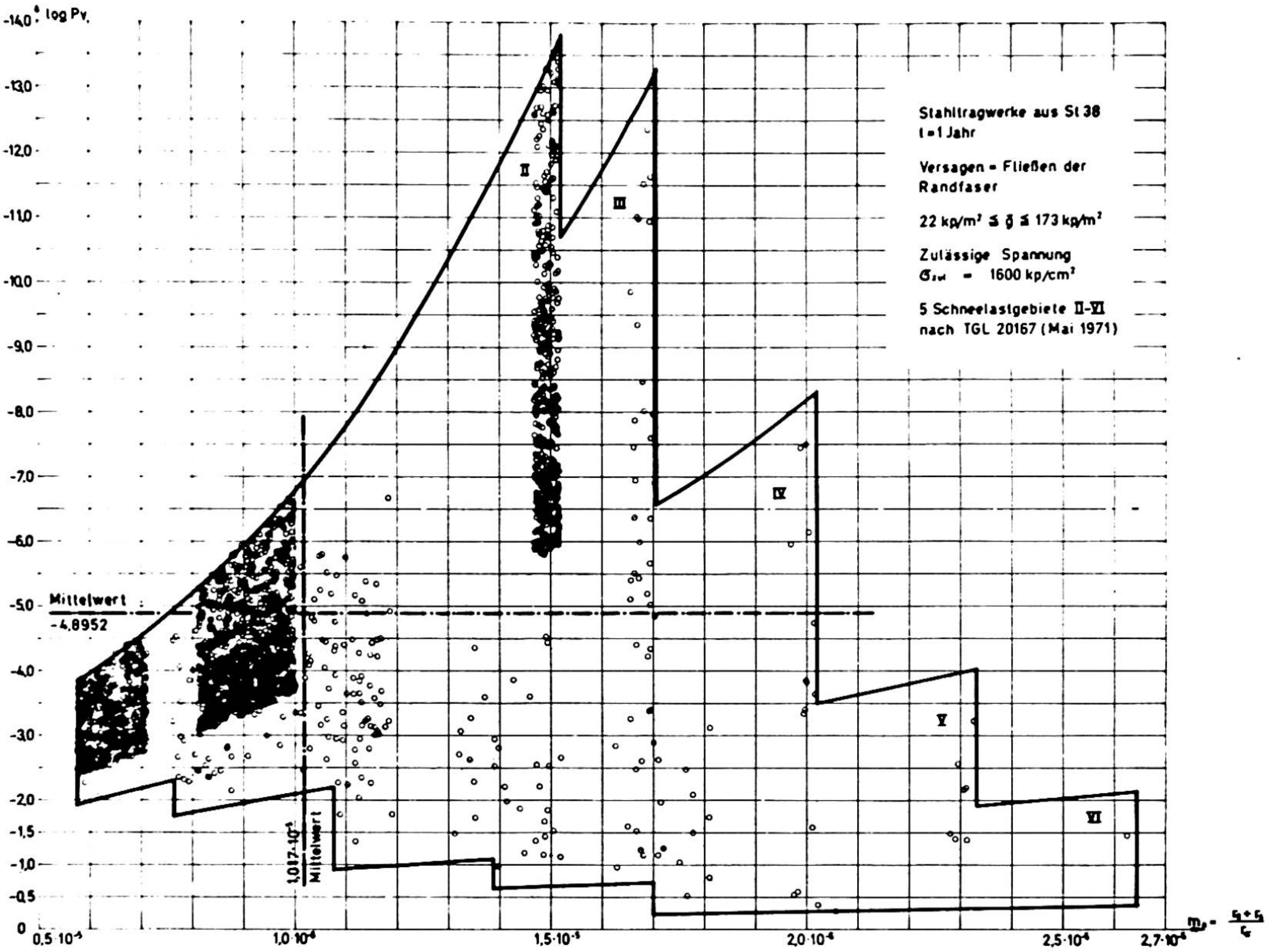

Bild 1:
Variationsgebiet, in dem die Versagenswahrscheinlichkeit Pv (Stahlfließen) und der bezogene Materialaufwand m_a (proportional den Querschnittswerten) beim z.Z. geltenden Vorschriftensystem der DDR liegen kann. Die Ergebnisse der Berechnungen von 2ooo für die Verhältnisse in der DDR repräsentativer Dächer sind als Punkte eingetragen.

Auf diese Weise wurden veraltete Normen, z.Z. gültige Normen und Vorschläge für eine Neufassung nach der Methode der Grenzzustände analysiert. Folgende Ergebnisse sind bemerkenswert:

- Die Versagenswahrscheinlichkeit in den unsicheren Grenzbereichen wird - historisch gesehen - immer kleiner.
- Durch eine in der DDR vor Jahren eingeführte Erhöhung der Schneelastannahmen in Gebirgslagen (ab 3oo m ü. NN) vermindert sich das Versagensrisiko um ca. 3o %.
- Die Einführung der Methode nach Grenzzuständen (konstanter Grenzlastfaktor 1,4 oder variabler Grenzlastfaktor 1,4 - 1,6, RGW-Standard St. 119-74) führt zu einer weiteren Erhöhung der Sicherheit.
- Das höhere Sicherheitsniveau wird aber mit mehr Material erkauft (z.B. Mehrverbrauch an Stahl durch höhere Schneelasten 1,6 % durch Grenzlastfaktor 2,2 %).

Um diesen Mehraufwand zu vermeiden, wurden Methoden zur Berechnung der Lastannahmen in den Vorschriften entwickelt, die eine gleichmäßigere Versagenswahrscheinlichkeit garantieren. Das angestrebte und auch erreichte Ziel dieser Untersuchungen war es, das erwünschte höhere Sicherheitsniveau in den unsicheren Grenzbereichen bei einem im Mittel über alle Tragwerke geringerem Materialaufwand zu erreichen.

Dazu wurde nachstehende Änderung für die Belastungsannahmen vorgenommen [3]:

- Einführung des Schneelastgebietes I mit einer Normlast von 5o kp/m^2 (an Stelle von bisher 7o kp/m^2) für das Flachland, wo ausreichende Sicherheitsreserven vorhanden sind.
- Berücksichtigung des starken Einflusses des Verhältnisses von Eigenlast zu Schneelast durch einen Kombinationsfaktor.
- Erhöhung der rechnerischen Schneelast bei Standorten in sehr hoher Lage über NN.

2. Gasexplosionen

Internationale und eigene Erfahrungen zeigen, daß der Verhütung größerer Schäden infolge Gasexplosionen im Wohnungsbau und der Vermeidung der damit verbundenen Gefährdung von Menschenleben größte Aufmerksamkeit zu widmen ist.

In der DDR sind hierzu umfangreiche theoretische und experimentelle Untersuchungen durchgeführt worden, deren Ergebnisse in einer dem derzeitigen Erkenntnisstand entsprechenden Projektierungsrichtlinie [4] zusammengefaßt sind. Sie bestätigen die Sicherheitsannahmen in den Bauvorschriften anderer Länder.

Wir orientieren uns dabei zunächst auf die Plattenbauweise im Wohnungsbau, um die hier mögliche Gefahr eines progressiven Kollapses weitestgehend auszuschließen. Bei ein- und zweigeschossigen Wohngebäuden sind nur allgemeine Grundsätze zur Reduzierung der Belastungsparameter zu berücksichtigen. Gleiches gilt für mehr- und vielgeschossige Wohngebäude, wobei aber die Belastungsparameter so zu beeinflussen sind, daß ein Belastungsprodukt von o.o25 (ats) nicht überschritten wird (vergleiche Bild 2). Bei größeren Werten wird ein Nachweis der Tragkonstruktion ohne Ansatz von Sicherheitsbeiwerten erforderlich. Bei Wohnhochhäusern sind Innenküchen mit Gas-Installation nicht zulässig.

Um die vom Druckimpuls betroffenen Bereiche der Tragkonstruktion so klein wie möglich zu halten und die eventuell in den Nachbarräumen befindlichen Personen nicht zu gefährden, muß das explodierende Gas-Luft-Gemisch auf dem kürzesten Wege nach außen gelangen können.

Experimente und weitere ingenieur-theoretische Betrachtungen führten zu der Erkenntnis, daß in Abhängigkeit von der raumabschließenden Konstruktion folgende Grundregeln bei der Projektierung im Wohnungsbau zu beachten sind, um den Belastungsimpuls möglichst gering zu halten:

- Anordnung großer Entlastungsflächen mit geringer Berstlast, die im Gefahrenfall zu Bruch gehen soll (z.B. Fenster, Türen und leichte Trennwände);
- Beschränkung der Gasinstallation auf nur einen Raum (Küche oder Bad);
- Vermeidung von langgestreckten oder abgewinkelten Explosionsräumen und Verhinderung der Gasausbreitung in mehreren Räumen.

In Fällen, in denen die kritische Grenze des Belastungsimpulses überschritten und eine zusätzliche Bemessung der Tragkonstruktionen erforderlich wird, muß sie sich auf solche Elemente

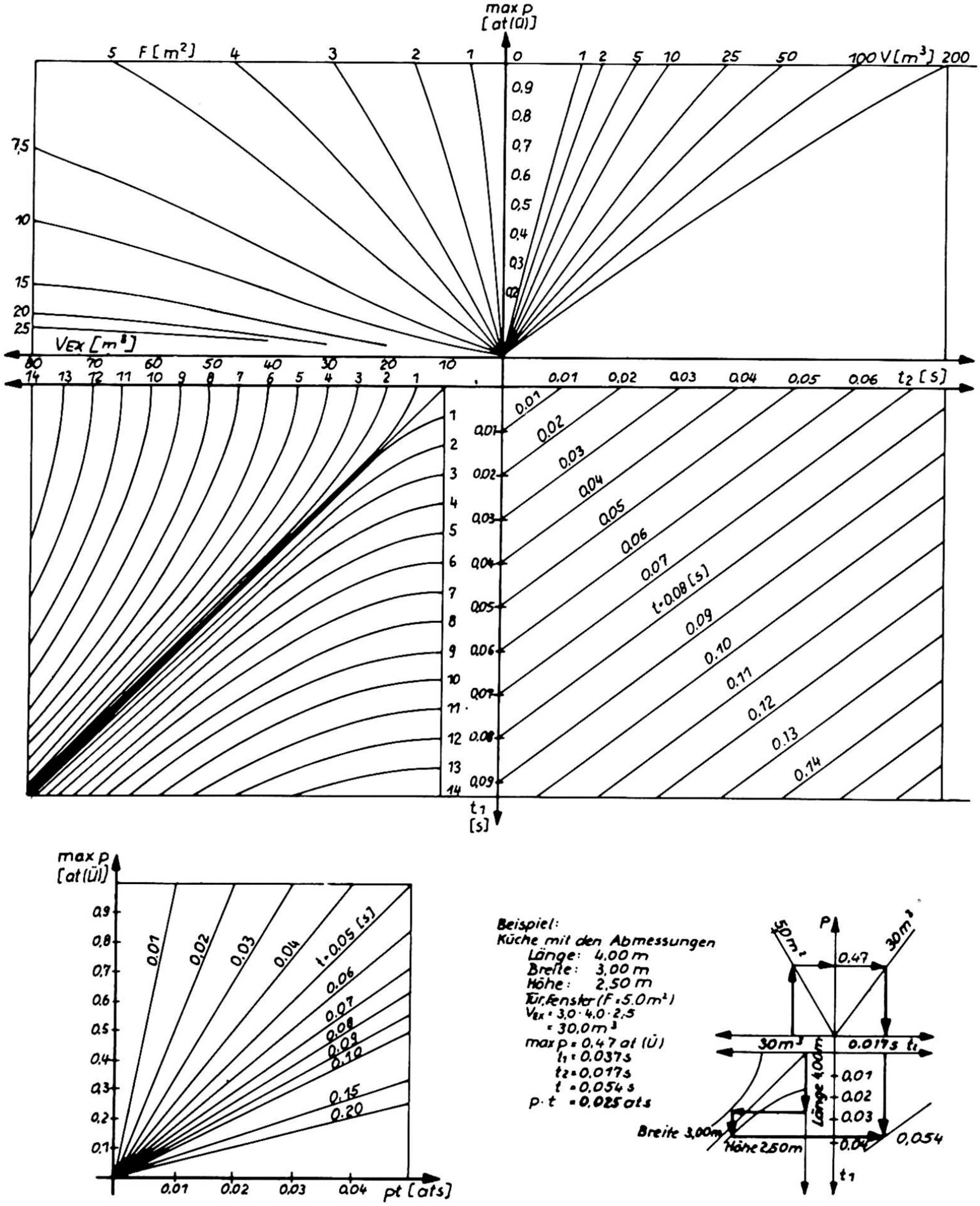

Bild 2: Diagramm zur Ermittlung des Belastungsimpulses aus Gasexplosionen in Wohnungen in Abhängigkeit vom Volumen des Explosionsraumes und der Entlastungsfläche

erstrecken, deren Versagen zu Schäden größeren Ausmaßes führen kann. Dabei ist der auf die Konstruktion wirkende Belastungsimpuls als eine Funktion der möglichen Druckentlastungsfläche und des Explosionsraumes zu ermitteln. Die konstruktive Durchbildung der Anschlüsse der tragenden Wand zur Aufnahme von Horizontalkräften muß besonders sorgfältig erfolgen, vor allen Dingen bei Giebelwänden. Die Zerstörung von Trennwänden, nichttragenden Außenwänden udgl., die für die Stabilität des Gesamtgebäudes nicht entscheidend sind, kann zugelassen werden [5].

Zusammenfassend gilt als Grundprinzip unserer Überlegungen:

- Gasexplosionen durch Einhaltung der sicherheits-technischen Forderungen zu vermeiden;
- mögliche Explosionsbelastungen für die Tragkonstruktionen durch ausreichende und funktionsfähige Druckentlastung gering zu halten.

Damit wird die erforderliche Sicherheit in der Regel ohne zusätzlichen materiellen Aufwand durch entsprechende funktionell-technische Entwurfslösungen erreicht.

3. Einbeziehung experimenteller Untersuchungen

Die aus dem Modellcharakter von Berechnungen resultierenden Grenzen ihrer Übereinstimmung mit der Wirklichkeit dürfen bei Sicherheits- und Zuverlässigkeitsbetrachtungen nicht außer acht gelassen werden. Infolgedessen betrachten wir experimentelle Untersuchungen zur Klärung rechnerisch nicht oder nur unsicher erfassbarer Zusammenhänge und eine Vereinfachung der Berechnungsmethoden als notwendig. Das gilt besonders dann, wenn es sich um Elemente oder Konstruktionen handelt, die in großer Stückzahl gefertigt werden und bei denen sich im Hinblick auf den großen Anwendungsumfang der oft erhebliche experimentelle Aufwand lohnt. Zwei Beispiele sollen dieses im Prinzip erläutern.

3.1. Großversuche an Elementegruppen von Gebäudeteilen im Maßstab 1 : 1

Solche Großversuche laufen z.Z. mit vorgespannten Deckenelementen des Wohnungs- und Gesellschaftsbaus. Sie sollen Auskunft über die zweckmäßigste Bewehrungsführung in diesen Deckenplatten unter Berücksichtigung aller Randbedingungen des vorhandenen statischen Systems geben und klären, ob der große Anteil an konstruktiver Zusatzbewehrung ohne Einschränkung der Sicherheit und Zuverlässigkeit reduziert werden kann. Deshalb werden nicht wie bisher üblich einzelne Deckenplatten geprüft, sondern ein aus Innen- bzw. Außenwänden und Decken zusammengesetztes Gebäudeteil mit den der Wirklichkeit entsprechenden Lasten und den aus der Bauausführung folgenden Randbedingungen.

Experimente an Decken werden auch unmittelbar in der Produktion unter Fertigungs-, Transport- und Montagebedingungen durchgeführt, um den Einfluß dieser nicht unwichtigen Zustände zu studieren. Zielstellung ist die Prüfung der Zuverlässigkeit, die Optimierung der Bewehrung bei Senkung des Stahlaufwandes und gleichzeitig eine optimale Gestaltung des Bewehrungsbaus (vergleiche Bild 3).

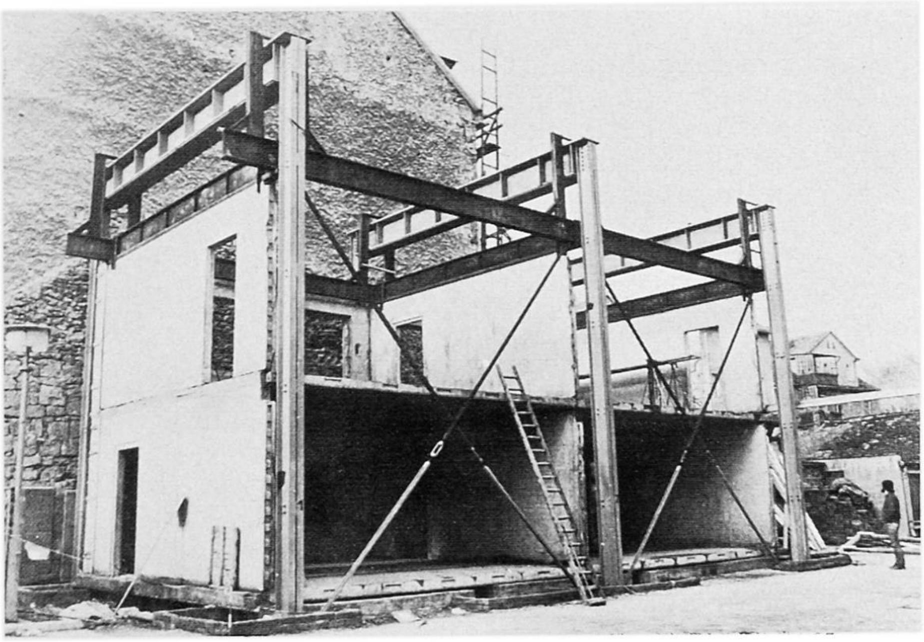

Bild 3:
Versuchsstand für Großversuche an Decken der Plattenbauweise

3.2. Reduzierung des Berechnungsaufwandes

Bei normalen Erzeugnissen großer Serien, zu denen wir den industriellen mehrgeschossigen Wohnungsbau rechnen, gehen wir dazu über, theoretisch und experimentell begründete Regeln den Projektanten vorzugeben, bei deren Erfüllung auf einen statischen Nachweis verzichtet werden kann. So konnten z.B. für die Aussteifung von Gebäuden bis zu 6 Vollgeschossen Bedingungen festgelegt werden, bei deren Einhaltung der rechnerische Nachweis der Stabilisierung gegen alle horizontalen Lasten, also auch Wind, entfallen darf. Im Prinzip kann dieser Nachweis mit zwei einfachen Formeln für die Längs- bzw. Querrichtung erbracht werden, in die außer geometrischen Werten (Scheibenhöhe, Geschoßhöhe, Querwandabstand, Gebäudebreite u.a.) nur die Eigenlasten der Decken und Wandflächen eingehen. Es wird auch angegeben, für welche Schubkraft T vorhandene Konstruktionselemente zu bemessen sind und in welchen Fällen eine Stabilisierung in Gebäudelängsrichtung ohne rechnerische Nachweise durch Außenwände allein übernommen werden kann. Die Entwicklung derartiger theoretisch und experimentell abgeklärter Näherungsverfahren bietet sich besonders für die Projektierung und Ausführung von Gebäuden aus industriell vorgefertigten Bauelementen und Baugruppen an. Der Ingenieur gewinnt dadurch Zeit, sich intensiver mit der fertigungs- und nutzungsgerechten sowie ökonomischen Durchdringung der Konstruktionen zu befassen, was mit zu einer seiner vordringlichen Aufgaben gehört.

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ZUSAMMENFASSUNG

Die Versagenswahrscheinlichkeit schwankt im Falle von Flachdächern mit Schneebelastung z.Z. in weiten Grenzen. Der Schwankungsbereich lässt sich bei gleichzeitiger Verringerung des Materialaufwandes durch eine Aenderung der Belastungsvorschriften einengen. An weiteren Beispielen wird der besondere Wert experimenteller Untersuchungen zur Erzielung eines ausgewogenen Verhältnisses zwischen Zuverlässigkeit und Aufwand bei Serienbauten aus industriell gefertigten Elementen aufzeigen.

SUMMARY

The probability of failure of flat roofs with snow loads varies at present within wide limits. These limits and the expenditure of materials can be reduced by a modification of load specifications. Some further examples demonstrate the great value of experimental studies for obtaining a well-balanced relation between reliability and expenditure of serial buildings in prefab construction.

RESUME

Dans le cas des toits plats sous la charge de la neige, la probabilité de ruine varie actuellement dans de larges limites. Une modification des prescriptions de charge, en même temps qu'une diminution de la quantité des matériaux permettent de réduire ces limites. Des exemples montrent la valeur particulière d'expériences permettant d'obtenir un rapport équilibré entre la sécurité et l'économie dans le cas de bâtiments préfabriqués avec des méthodes industrialisées.

Anwendung der stochastischen Programmierung für die Berechnung der Sicherheit und für die Optimierung von Konstruktionen

Application of Stochastic Programming for the Computation of Safety and for the Optimization of Structures

Application de la programmation stochastique pour le calcul de la sécurité et pour l'optimisation des structures

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1. Einleitung

Die Beurteilung der Sicherheit statisch unbestimmter Konstruktionen ist wegen der Möglichkeit der Spannungsumlagerung nicht in gleicher Weise möglich wie bei statisch bestimmten Konstruktionen. Als wesentlicher Parameter zur Beurteilung der Sicherheit gilt die Versagenswahrscheinlichkeit, das heißt, die Wahrscheinlichkeit, daß eine genau definierte Grenzlast eines Tragwerks überschritten wird. Die deterministische Berechnung der Grenzlast erfolgt auf der Grundlage der Traglastsätze /¯1_7. Die Anwendung des zweiten Traglastsatzes zur Bestimmung der Versagenswahrscheinlichkeit statisch unbestimmter Stahlrahmen wurde von F.Moses /¯2_7 gezeigt. Bei diesem Verfahren müssen alle kinematisch verträglichen Verschiebungszustände (kinematische Ketten,"Failure Modes") angegeben werden. Im vorliegenden Beitrag wird nun vorgeschlagen, die Versagenswahrscheinlichkeit aus der systematischen Formulierung des Traglastproblems als mathematische Programmierungsaufgabe /¯1_7 mit Hilfe von Verfahren aus der stochastischen Programmierung zu berechnen. Die Erweiterung der Bemessungsaufgabe, formuliert als plastische Optimierung, auf eine Bemessung für eine zulässige Versagenswahrscheinlichkeit der Gesamtkonstruktion folgt dann aus dieser stochastischen Traglastberechnung.

2. Traglastberechnung mit linearer stochastischer Programmierung

Eine lineare deterministische Formulierung des Traglastproblems ist durch

$$\text{maximiere } \lambda$$

unter den Nebenbedingungen

$$R\,(\,\lambda b_o P + b_x X\,) \geq F_o \qquad (1)$$
$$\lambda \geq 0$$

gegeben /¯1_7. Hierbei bedeutet :

λ : Traglastfaktor ;

R : (p,n)-Matrix, deren Koeffizienten sich aus der Linearisierung der nichtlinearen Fließbedingungen ergeben; mit p = r k, das ist : Anzahl der Gleichungen für eine linearisierte Fließbedingung (r) mal Anzahl der Kontrollpunkte, in denen Fließbedingungen aufgestellt wurden (k); n ist die Anzahl der Schnittkräfte;

b_o : (n,m)-Matrix der Einheitsspannungszustände des statisch bestimmten Hauptsystems mit m als Anzahl der Gleichgewichtsbedingungen;

P : (m)-Vektor der Knotenlasten;
b_x : (n,n-m)-Matrix der Einheitsspanunngszustände aus den statisch Unbestimmten;
X^x : (n-m)-Vektor der statisch Unbestimmten;
F_o : (p)-Vektor der rechten Seiten der linearisierten Fließbedingungen.

Die Anzahl der Unbekannten ist $h = n - m + 1$.

Faßt man die Unbekannten im (h)-Vektor y und die Koeffizienten der Restriktionen in der (p,h)-Matrix B zusammen, ergibt sich mit $c'=(1,0,\dots,0)$ als (h)-Vektor der Kostkoeffizienten die Standardformulierung einer linearen Optimierungsaufgabe :

$$\text{maximiere} \quad c'y$$

unter den Nebenbedingungen

$$\begin{aligned} B\,y &\geqslant F_o \\ y_1 &> 0 \quad . \end{aligned} \tag{2}$$

Die hierzu duale Formulierung lautet :

$$\text{minimiere} \quad F_o'\,z$$

unter den Nebenbedingungen

$$\begin{aligned} B'\,z &= c \\ z &> 0 \quad . \end{aligned} \tag{3}$$

Von M.M.Faber [3] wird der Einfluß stochastischer Größen in der Koeffizientenmatrix B oder in den Vektoren c und F_o auf den Wert der Zielfunktion untersucht.

Bei den hier betrachteten Traglastproblemen enthält die erste Spalte von B mit den Knotenlasten P stochastische Variable; die übrigen Elemente von B sind aus den Systemabmessungen abgeleitet und werden wegen ihres kleinen Streubereichs als fest vorgegeben betrachtet. Der Vektor F_o enthält mit den Festigkeiten der Werkstoffe (z.B.der Fliesspannung σ_F) ebenfalls stochastische Größen. In Abhängigkeit dieser stochastischen Größen ergibt sich eine Verteilungsfunktion $F(\lambda)$ für den Lastfaktor λ . Die Versagenswahrscheinlichkeit ist dann gegeben durch

$$p_f = W(\lambda \leqslant 1) = F(1) \quad . \tag{4}$$

Eine vereinfachte Möglichkeit der Berechnung der Versagenswahrscheinlichkeit ergibt sich, wenn man für proportionale Belastung, das heißt, die Verhältnisse der Knotenlasten zueinander bleiben konstant, bei der stochastischen Lösung von Problem (2) oder (3) nur die Verteilung der Elemente von F_o berücksichtigt. In einem zweiten Rechengang kann dann die Verteilung der Last mit der Verteilung des Lastfaktors verknüpft werden. Berechnet man die Versagenswahrscheinlichkeit näherungsweise mit der Methode der zweiten Momente [4], so genügt es, den Erwartungswert und die Varianz des Lastfaktors zu bestimmen.

Setzt man für die Elemente von F_o deren Erwartungswerte ein, so ergibt sich, wie bei der deterministischen Berechnung vorausgesetzt wird, der Erwartungswert von λ .Nach [3] läßt sich bei einer Lösung von Problem (2) die Varianz des Lastfaktors aus

$$\sigma_\lambda^2 = s'\,C_F\,s \quad , \tag{5}$$

oder bei einer Lösung von Problem (3) aus

$$\sigma_\lambda^2 = z'\,C_F\,z \tag{6}$$

errechnen. Hierbei bedeutet :

C_F : Kovarianzmatrix der Elemente von F_o ,
s : Vektor der Simplexkoeffizienten; sie werden bei einer Lösung nach der Simplexmethode benötigt ,
z : Lösungsvektor von Problem (3).

Der Streubereich der stochastischen Variablen F_o darf allerdings nur so groß sein, daß weder die Zulässigkeit noch die Optimalität der Lösung verlorengeht. Der maximal zulässige Streubereich ergibt sich aus einer Sensitivitätsanalyse [5].

3. Berechnung der Versagenswahrscheinlichkeit nach der Methode der Momente

Sind der Mittelwert $\bar{P}$ und die Streuung σ_P der Last gegeben, so kann mit der Methode der zweiten Momente [4] Erwartungswert und Varianz des Lastfaktors zur Berechnung der Versagenswahrscheinlichkeit verwendet werden.

Die Sicherheitszone ist gegeben durch

$$Z = (\lambda - 1)\ \bar{P} \quad , \tag{7}$$

ihre Varianz durch

$$\sigma_Z^2 = \sigma_\lambda^2 + \sigma_P^2 \, . \tag{8}$$

Die Versagenswahrscheinlichkeit ist dann

$$p_f = \varphi(-\frac{Z}{\sigma_Z}) \, . \tag{9}$$

φ ist die normierte Gauß'sche Verteilungsfunktion.

4. Plastische Optimierung für eine zulässige Versagenswahrscheinlichkeit

Die deterministische Formulierung des plastischen Bemessungsproblems ist

$$\text{Min} \left\{ \sum_{i=1}^{e} \gamma \cdot l_i A_i \middle| \Phi_j(X,A_i) \geq 0, A_i \geq 0 \right\} \quad \begin{matrix} j = 1 \ldots k, \\ i = 1 \ldots e. \end{matrix} \tag{10}$$

Die Querschnittsflächen A_i und die Eigenspannungszustände X sind die Variablen des Problems; e ist die Anzahl der Elemente. Gesucht ist also das minimale Gewicht des Gesamttragwerks bei Einhaltung der Fließbedingungen Φ_j für einen fest vorgegebenen Traglastfaktor λ_o Die Schnittkräfte wurden bei (17) durch die statisch Unbestimmten und den konstanten Anteil $\lambda_o b_o P$ ausgedrückt.

Um den stochastischen Parametern in Problem (17), Belastung und Festigkeit, Rechnung zu tragen, muß man bei der plastischen Optimierung noch die Versagenswahrscheinlichkeit berücksichtigen. Das Problem lautet dann:

$$\text{Min} \left\{ \sum_{i=1}^{e} \gamma l_i A_i \middle| \Phi_j(\lambda, X, A_i) \geq 0, p_{f\,zul} - p_f \geq 0, A_i \geq 0 \right\} \tag{11}$$

$p_{f\,zul}$ ist die zulässige Versagenswahrscheinlichkeit.

Der Traglastfaktor λ gehört bei diesem Problem zu den Variablen.

Berechnet man die Versagenswahrscheinlichkeit nach der Methode der zweiten Momente, so kann die Wahrscheinlichkeitsrestriktion nach einem Vorschlag van Bracken und Mc Cormick [6] in ein deterministisches Äquivalent umgeformt werden.

$$p_f = \varphi(-\frac{Z}{\sigma_Z}) \geq p_{f\,zul} \tag{12}$$

Mit der inversen Gauß'schen Verteilungsfunktion φ^{-1} gilt dann:

$$\varphi^{-1}(p_{f\,zul}) + \frac{Z}{\sigma_Z} \geq 0 \tag{13}$$

oder

$$\varphi^{-1}(p_{f\,zul})\,\sigma_Z + Z \geq 0 \quad . \tag{14}$$

Bei vorgegebener zulässiger Versagenswahrscheinlichkeit ist $\varphi^{-1}(p_{f\ zul})$ eine Konstante mit gleicher Dimension wie die Sicherheitszone Z. Zur Bestimmung der Varianz des Lastfaktors λ nimmt man näherungsweise eine lineare Funktion in den stochastischen Variablen an. Für das Testbeispiel, bei dem nur die Varianz der Fliesspannung σ_F berücksichtigt wurde, wurde

$$\lambda_{opt} = k\ \sigma_F \qquad (15)$$

gesetzt. Somit gilt für die Streuung

$$\sigma_\lambda = \sqrt{k}\ \sigma_\sigma \qquad (16)$$

Die Streuung der Sicherheitszone ist dann durch (8)gegeben.

5. Testbeispiele

Das beschriebene Verfahren zur Berechnung von Versagenswahrscheinlichkeiten wurde an zwei Konstruktionen getestet.

5.1. Fachwerk

Die Fließbedingung für Fachwerkstäbe lautet:

$$F \geqslant F_o \ ,$$

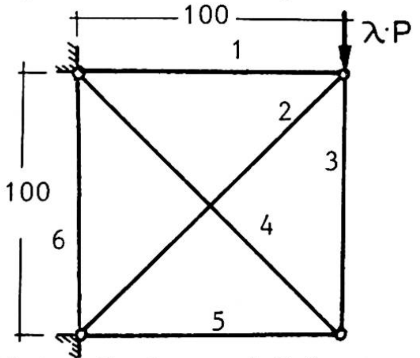
$$-F \geqslant F_o \ .$$

Somit ergibt sich für die Matrix R

$$R = \begin{bmatrix} I \\ -I \end{bmatrix} .$$

I ist die (n,n)-Einheitsmatrix .

Systemabmessungen und Belastung sind in Bild 1 dargestellt.



Querschnittswerte :

$A_1 = 0.6462 \ [\mathrm{cm}^2]$

$A_2 = 0.7996 \ [\mathrm{cm}^2]$

$A_3 = A_4 = A_5 = A_6 = 0.1 \ [\mathrm{cm}^2]$

Belastung : $\bar{P} = 1000 \ [\mathrm{kp}]$

Variationskoeffizient der Last: $v_P = 0.1$

Bild 1 : System und Belastung des Fachwerks

Die Querschnittswerte sind mit dem Optimierungsverfahren von W.Lipp und G.Thierauf [7] ermittelt worden. Als zulässige Spannungen wurden hierbei eingesetzt :

$$\text{Druck :} \quad \sigma_{zul} = 1400 \left[\frac{\mathrm{kp}}{\mathrm{cm}^2}\right] ,$$

$$\text{Zug :} \quad \sigma_{zul} = 1600 \left[\frac{\mathrm{kp}}{\mathrm{cm}^2}\right] .$$

Die vollplastischen Schnittgrößen F_o für die Traglastberechnung wurden mit einem Mittelwert der Fliesspannung $\bar{\sigma}_F$ berechnet.

$$F_o = A_i \cdot \bar{\sigma}_F \ , \quad \text{im Beispiel} \quad F_o = A_i \cdot 2700 \ .$$

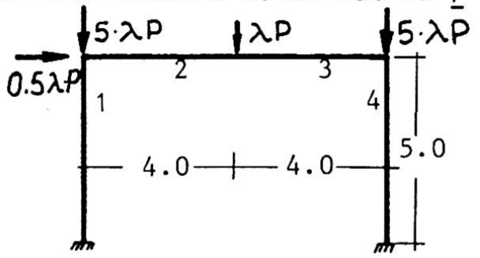
Die Berechnung der Versagenswahrscheinlichkeit erfolgte somit für einen Traglastfaktor $\lambda = 1.5267$. Für unterschiedliche Variationskoeffizienten der Fliesspannung v_σ ist das Ergebnis in Tabelle 1 zusammengestellt.

v_σ	σ_λ	P_f
0.1	0.057	2.37 10^{-6}
0.2	0.114	2.57 10^{-4}

Tabelle 1 : Streuung des Lastfaktors und Versagenswahrscheinlichkeit des Fachwerks

5.2.Rahmen

Für die Traglastberechnung und die plastische Optimierung des in Bild 2 dargestellten Rahmens wurde das in /‾1_7 beschriebene Verfahren verwendet.



	λ_σ = 1.5		λ_σ = 1.7	
Element	MPL	NPL	MPL	NPL
1	0.59	7.12	0.66	8.1
2	2.34	28.40	2.65	32.19
3	2.24	27.21	2.54	30.84
4	1.48	18.00	1.68	20.40

MPL /‾ Mp m_7 : vollplastisches Moment

NPL /‾ Mp _7 : vollplastische Normalkraft

Mittelwert der Last : $\bar{P}$ =1.0 /‾Mp_7

Variationskoeffizient der Last : v_P = 0.1

Bild 2 : System und Belastung des Rahmens

Die Querschnittswerte (vollplastische Schnittgrößen MPL und NPL) wurden für eine Fliesspannung von

$$\sigma_F = 24000 \; /\overline{} \frac{Mp}{m^2} \_7$$

mit Hilfe der Traglastbemessung ermittelt (Plastic Design).Die Berechnung der Versagenswahrscheinlichkeit ergab für einen Mittelwert der Fliesspannung

$$\bar{\sigma}_F = 27000 \; /\overline{} \frac{Mp}{m^2} \_7$$

die in Tabelle 2 zusammengestellten Werte.

	λ = 1.6875		λ = 1.9125	
v_σ	σ_λ	P_f	σ_λ	P_f
0.05	0.045	1.71 10^{-10}	0.05	0.0
0.1	0.089	1.45 10^{-7}	0.10	6.2 10^{-11}
0.2	0.178	3.89 10^{-4}	0.201	2.4 10^{-5}

Tabelle 2 : Streuung des Lastfaktors und Versagenswahrscheinlichkeit des Rahmens

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ZUSAMMENFASSUNG

Es wird ein Verfahren zur näherungsweisen Berechnung der Versagenswahrscheinlichkeit statisch unbestimmter Konstruktionen mit Hilfe der stochastischen Programmierung angegeben. Aus dieser Berechnungsmethode folgt dann die Formulierung der Traglastbemessung (plastische Optimierung) für zulässige Versagenswahrscheinlichkeiten.

SUMMARY

A method for the approximate computation of the probability of failure of statically indeterminate structures is proposed. It represents a direct application of stochastic programming to the limit analysis problem. This solution then leads to a formulation of the plastic design problem for allowable probabilities of failure.

RESUME

On propose dans ce travail une méthode pour estimer la probabilité de ruine des structures hyperstatiques. La méthode est une application directe de programmation stochastique à l'analyse des charges limitées. Cette solution donne une formulation pour le problème du dimensionnement plastique avec une probabilité de ruine admissible.

Resistance of Steel Structures according to the Limit States Method

Résistance des structures en acier selon la méthode des états limites

Die rechnerische Tragfähigkeit von Stahlkonstruktionen nach der Methode der Grenzzustände

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1. Introduction

In Czechoslovakia, the method of limit states is used for the calculation of steel structures. The calculation basis corresponds with the calculation according to the International Standard of member states of the Council of Mutual Economic Aid. It is, however, adapted to Czechoslovak material and loading conditions.

The Czechoslovak standards defining the limit states of steel structures are as follows: ČSN 73 0031 Basic Regulations for the Calculation of Structures, ČSN 73 0035 Loading of Steel Structures, ČSN 73 1401 Design of Steel Struc ures. All these standards are being revised at present.

According to the standards the limit state of a structure is such a state, in which the structure loses its load-carrying capacity or it does not satisfy the demands its current use. Two groups of limit states are differentiated:

a/ limit states of the load-carrying capacity - the states in which the structure loses the load-carrying capacity, or in which further use of a structure is excluded,

b/ limit states of serviceability - the states in which the structure is no more appropriate for the common use, or in which its durability decreases.

The limit states are based on the probabilistic method of calculation. The method of mathematical statistics are used only for the determination of the initial quantities - the values of load and resistance of a material, or of a structure. As each quantity is analysed individually, the limit states method is a semi-probabilistic one.

1.1 Material resistance

The specified and factored resistance is distinguished. The specified resistance R_s is determined with respect to the method of checking the material properties and their variation. It ought to correspond to at least the probability value of 0,05. The possible unfavourable deviations of the structure resistance from the specified values are expressed by the factored resistance R. Its definition will be quoted later. The ratio of the specified resistance and the factored resistance is material factor K.

1.2 Loading of structures

Similarly, the specified, factored load and the load factor are considered for loading. Besides,a load combination factor also is applied.

These problems are not dealt with here and thus no standard definitions are stated.

2. Statistic nature of limit states

If the i-th load effect is denoted by S_i and the structure resistance by R^x, the safety margin G is defined as

$$G = R^x - \sum_i S_i . \qquad /1/$$

The limit state occurs when the safety margin is exceeded, hence if $G = 0$.

The load effects and structure resistance are random variable, mostly mutually independent quantities, and thus the safety margin has to be studied statistically and the limit state has to be determined for some small probability of occurrence.

These problems were studied by A.R. RZHANICYN [12], [13] and some other authors later on - i.g. [1], [2], [4], [11].

Let us pay attention to the main theoretical results and show the transition from them to the present limit states method.

The statistical characteristics [14] of difference /1/ of independent randomly variable quantity are:
mean value

$$m_G = m_R - m_S, \qquad /2/$$

standard deviation

$$s_G = \sqrt{s_R^2 + s_S^2} , \qquad /3/$$

skewness

$$a_G = \frac{s_R^3 a_R - s_S^3 a_S}{\left(\sqrt{s_R^2 + s_S^2}\right)^3} . \qquad /4/$$

Then, the safety margin G for a certain probability p is

$$G_p = m_G - \beta_{pa_G} s_G = m_R - m_S - \beta_{pa_G} \sqrt{s_R^2 + s_S^2} \geqq 0. \qquad /5/$$

Using LIND's [4] separation linearisation function α_{RS} this inequality can be written in the form

$$m_R \left(1 - \alpha_{RS} \beta_{pa_G} v_R \right) \geqq m_S \left(1 + \alpha_{RS} \beta_{pa_G} v_S \right) , \qquad /6/$$

where $v_R = s_R/m_R$, $v_S = s_S/m_S$.

Condition /6/ formally separates the structure resistance from the load effects. In practical cases the distribution functions of quantities R and $\sum S$ are often considered sufficiently accurately according to the normal distribution ($a_R = a_S = 0$). Besides if the separation linearisation function α_{RS} is considered as a constant value according to LIND, then the separation is complete.

The Czechoslovak limit states method considers $\alpha_{RS} \approx 1{,}0$ and studies separately the structure resistance and the load effects.

The structure resistance is expressed in terms of the material characteristics with the dimension of stress and is referred as factored resistance R.

The mathematically statistical representation of resistance R by help of the left side of the inequality /6/, is

$$R = m_R (1 - \beta_{pa_R} \; v_R), \qquad /7/$$

where only skewness a_R is considered.

3. Factored resistance of steel structures

The resistance of steel structures is derived from the yield point stress of steel σ_y. The specified resistance R_s is a statistical value of the yield point stress for probability $p = 0{,}05$.

When considering the material, the structure safety is affected not only by yield point stress, but also by random dimensions of cross-sections, by their deformation due to manufacture, by technological effects, etc. The steel behaviour in a structure is not the same as the behaviour during a tension test. The yield point stress in a cross-section is not the same as in the place from which the test specimen is taken. The test values are affected by the size and the shape of test speciment and by the testing method.

All these effects are randomly variable quantities. Only some of them can be investigated separately /e.g. size of cross-section/. The other influences are referred to as unknown imperfections. Thus all the effects are generally expressed by the conventional randomly variable cross-section area and a uniform statistical model of a tension bar and the probability of $p = 0{,}001$ is taken into consideration.

Let the real yield point stress be $[\sigma_y]_i$, the imperfections represented by the area $[A]_j$ and the theoretical area A, then the condition of tension is $\bar{R}A = [\sigma_y]_i \, [A]_j$, or

$$\bar{R} = [\sigma_y]_i \, \frac{[A]_j}{A} = [\sigma_y]_i \; [f]_j \; . \qquad /8/$$

It is the product of two independent randomly variable quantities.

The definition of the factored resistance is:
The factored resistance R is the boundary value $\bar{R}_{0,001}$ of the product /8/ for probability $p = 0{,}001$.

Using the PEARSON's density curve of type III expressed by functional relations according to VORLÍČEK [14] we can write

$$R = \bar{R}_{0,001} = m_R - \beta_{0,001 \, a_R} \, s_R =$$
$$= m_y \, m_f (1 - \beta_{0,001 \, a_R} \sqrt{v_y^2 + v_f^2 + v_y^2 \, v_f^2} \;) , \qquad /9/$$

where m_R, s_R are the statistic characteristics of product /8/,

m_y, s_y - the statistic characteristics of yield point stress σ_y,

m_f, s_f - the statistic characteristics of ratio f,

$\beta_{0,001 \, a_R}$ - the quantile of probability $p = 0{,}001$ depending upon the skewness of distribution of density

$$a_R = \frac{v_y^3 \; a_y + v_f^3 \, a_f + 6 \, v_y^2 \, v_f^2}{(v_y^2 + v_f^2 + v_y^2 \cdot v_f^2)^{3/2}} \, .$$

The Author determined the numerical values of the factored resistance R according to the test results of steels in steel works in the years 1956-1959, 1966-1969 and 1971-1975. The values from the first period served for the preparation of the Czechoslovak Standard ČSN 73 1401, the values from following periods for the verification and refinement of factored resistances in this standard and for its revision.

The statistical analysis was carried out not only for the yield point stress σ_y, but also for the strength $\sigma_{T.S.}$, the ductility δ, the brittleness /results of Charpy tests/ and the ratio of areas of cross-sections. The results of the years 1956-69 are published in [5] to [10]. The recent results are prepared for publication. Some of these results are given here.

In Tab. 1 the statistical characteristics for common steels are given. Tab. 2 gives the boundary values of the yield point stress and of strength and factored resistances. The statistic values of the factored resistance R_{stat}, of the boundary yield point stress $\sigma_{y\,0,05}$ and the strength $\sigma_{T.S.\,0,05}$ allow a comparison with the standard values.

4. Conclusion

The paper shows that the limit states method allows the analysis of structure resistance and load effects. Even though it does not fully use the safety margins of structures, it characterizes the behaviour of structure more accurately than the method of allowable stresses based on the total safety factor.

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Table 1: Mean values m, standard deviations s, skewnesses a of yield point stress σ_y and tensile strength $\sigma_{T.S.}$

Steel Designation	Product, thickness	mm	Standard $\sigma_{T.S}/\sigma_y$ N/mm2	Yield point stress: Number of results	m_y N/mm^2	s_y N/mm^2	a_y	Tensile strength: Number of results	$m_{T.S.}$ N/mm^2	$s_{T.S.}$ N/mm^2	$a_{T.S.}$
11 373.0	Plates	>3-13	370/240	24 915	272,6	27,0	0,897	25 889	412,2	32,6	0,313
11 373.1		14-25	370/240	4 105	268,0	23,3	0,901	4 098	409,1	30,8	0,447
		26-60	370/230	1 242	266,0	26,0	0,366	1 216	414,2	31,9	0,713
11 373.0	Angles	≤ 13	370/240	8 983	273,2	21,5	0,488	8 504	406,2	29,6	0,338
	Beams	≤ 13	370/240	6 521	283,8	24,3	0,535	6 617	418,2	30,4	0,320
11 483.1	Plates	26-50	480/360	1 526	391,1	23,6	0,241	1 522	555,3	28,4	-0,103
11 523.1	Plates	>3-16	520/360	8 115	386,7	29,8	-0,209	8 142	566,0	34,1	-0,145
		17-25	520/350	2 948	387,4	26,0	-0,165	3 654	571,2	34,1	-0,067
11 523.1	Flat	≤ 16	520/360	1 976	384,9	29,1	0,312	1 994	567,2	35,1	0,212
	steel	17-25	520/350	1 804	389,4	25,9	0,656	1 997	579,5	32,9	0,534
11 523.0	Angles	≤ 13	520/360	236	379,7	28,1	0,198	243	555,0	43,8	0,561
	Beams	≤ 13	520/360	160	394,2	27,1	0,149	147	563,9	35,1	0,151

Table 2: Boundary values of yield point stress σ_y and tensile strength $\sigma_{T.S}$ for probabilities 0,001, 0,05 and 0,999
Factored resistance R according to statistic evaluation and to Czechoslovak Standard ČSN 73 1401

Steel Designation	Product, thickness	mm	Yield points stress N/mm^2 σ_y 0,001	σ_y 0,05	σ_y 0,999	Tensile strength N/mm^2 $\sigma_{T.S.}$ 0,001	$\sigma_{T.S.}$ 0,05	$\sigma_{T.S.}$ 0,999	Factored resistance N/mm^2 R_{stat}	R_{CSN}	$\frac{R_{stat}}{R_{CSN}}$	$\frac{\sigma_{y\,0,05}}{R_{stat}}$
11 373.0	Plates	> 3-13	221,2	236,2	391,1	325,8	361,7	527,3	214,4	210	1,021	1,102
11 373.1		14-25	223,7	236,3	370,3	333,0	362,6	524,3	220,8	210	1,051	1,070
		26-60	199,2	226,2	360,1	346,6	368,9	545,6	197,9	200	0,990	1,143
11 373.0	Angles	≤ 13	222,1	241,2	354,9	328,6	360,6	512,2	208,8	210	0,992	1,155
	Beams	≤ 13	226,7	247,8	377,6	337,9	371,1	526,1	214,8	210	1,023	1,154
11 483.1	Plates	26-50	326,2	353,8	472,0	463,9	501,9	638,8	324,2	290	1,118	1,091
11 523.1	Plates	>3-16	285,7	336,0	469,8	453,5	508,7	668,6	280,8	290	0,968	1,197
		17-25	301,6	343,7	421,5	471,7	514,6	673,5	298,4	290	1,029	1,152
11 523.1	Flat	≤ 16	307,8	339,8	487,8	466,9	512,1	686,2	302,6	290	1,043	1,123
	steel	17-25	332,4	352,1	493,8	501,9	530,8	706,5	326,5	290	1,126	1,078
11 523.0	Angles	≤ 13	301,0	335,0	474,4	453,4	490,6	725,8	286,7	290	0,989	1,168
	Beams	≤ 13	316,4	350,6	483,6	463,2	507,4	679,7	300,5	290	1,036	1,167

SUMMARY

The statistical basis of limit states method and their definition according to Czechoslovak standards are adduced. The limit states investigate separately load effects and structure resistance. The statistical definition of factored resistance is pointed out. Statistical results are published.

RESUME

L'auteur met en relief la nature statistique de la méthode des états limites et de leur définition d'après les normes tchécoslovaques. Les états limites étudient les effets séparés des charges et la résistance des constructions. L'auteur présente une définition statistique de la résistance. Des résultats de recherches statistiques sont publiés.

ZUSAMMENFASSUNG

Es werden die statistischen Grundlagen der Grenzzustände-Methode und deren Definition nach den tschechoslowakischen Normen angegeben. In den Grenzzuständen werden die Lastwirkung und die Tragfähigkeit getrennt. Die statistische Definition der Tragfähigkeit wird dargestellt und entsprechende Ergebnisse werden veröffentlicht.

Dynamic Aseismic Design Procedure and Example of High-rise Building in Japan

Etude parasismique dynamique des bâtiments de grande hauteur et exemple japonais

Dynamische Berechnungsmethode der Erdbebenwirkung an einem Hochhaus in Japan

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1. ASEISMIC DESIGN PROCEDURE

In Japan, aseismic design code called "seismic load coefficient method", in which the coefficient of 0.1G was regulated, was first adopted after the KWANTO Earthquake of 1923. When the "Building Law" was passed in 1950, the coefficient was altered to 0.2G, with the increment along the height of buildings, according to the change in allowable stresses. This design method is used for buildings lower than 45 meters in height. Under this method, the component stresses, ie., the above-mentioned seismic load combined with dead and live load stresses, must be within the allowable limits.

Since the 1950's, there has been considerable research on a variety of problems related to high rise buildings, for example, the effects of elastic and elastic-plastic vibrations. These efforts have yielded a unique "Dynamic Aseismic Design Method", which has allowed construction of buildings in excess of 45 meters. Since 1963, when the Minister of Construction first approved this, it has become possible to erect safer high-rise buildings that are also low cost.

1-1. Dynamic Design Procedure

The flow chart in Fig.1 shows the above-mentioned new dynamic aseismic design procedure called "feed-back system";

1) Assume the preliminary seismic load and its distribution.
2) Make static stress analyses of the structure.
3) Determine the preliminary structural design (the conventional design is finished at this step).
4) Establish a mathematical model to simulate the behavior of the preliminary designed structure.
5) Select suitable input earthquake waves among recorded waves at locations having similar ground conditions in the past and amplify their intensities to several different levels such as that of severe earthquakes and hypothetically the worst earthquakes.
6) Carry out earthquake response analyses covering the elastic and/or elastic-plastic ranges.
7) Check response values (stresses and strains) according to the design criteira. (In usual office buildings, for expected severe earthquakes, the stresses on all members must be less than the allowable values; similarly, if the story drift is less than 2cm, and for hypothetically

the worst earthquake, the structure may suffer some damage but must not be severely damaged)

8) If the safety judges are unsatisfactory (NO), modify the design until a satisfactory result is achieved (YES).

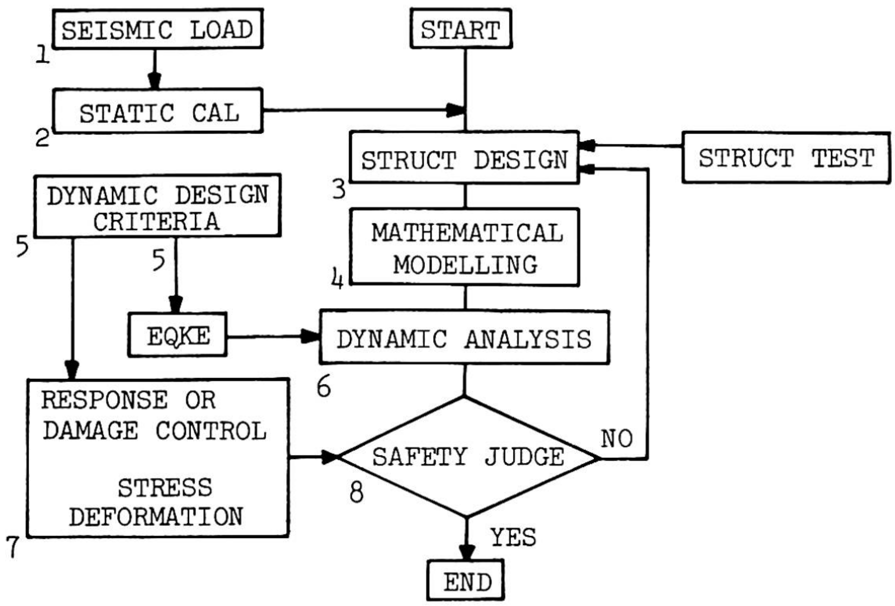


FIG.1 DYNAMIC DESIGN PROCEDURE

1-2. Dynamic Analyses

For dynamic analyses, the fundamental requirements are a rigorous vibration model and a computer program equipped to simulate as possible the actual behavior of the structure. To insure the adequacy of this process structural tests must be measured against the repeated lateral force and vibration results. Fig.2 shows structural tests on the steel frames and shearing walls. As shown in the lower part of this figure, the restorative qualities through both their elastic and elastic-plastic ranges serve as primary input date for the process known as FAPP-FASP system.

<u>FAPP</u> (Frame Analysis in consideration of Pure-shear Panel deformation)

FAPP is a computer program for rigorous analyses of high-rise frames such as open-, walled-, braced- and 3-dimensional frames as shown in Fig.3. In this FAPP method, it is assumed that framing is composed of four elements, ie., columns, beams, joint panels and braces, and bending, shearing and axial deformations on columns, bending, shearing and torsional deformations on beams, shearing deformations on joint panels, and axial deformation on braces are taken into account. Earthquake response analyses in the elastic range are easily carried out under this FAPP method as a coupled system for various kinds of frames.

<u>FASP</u> (Frame Analysis by Simplified Procedure)

FASP is a modified method especially for non-linear analyses as shown in Fig.4. In this method, a non-linear hysteresis loop for the bending and shearing rigidities of the FASP model (lumped-mass system) are evaluated by placing the results of step-by-step analyses of FAPP in elastic and elastic-plastic ranges, against the static, gradually increasing lateral force.

2. DESIGN EXAMPLE (55-story Office Building)

The design example introduced here is the Shinjuku Mitsui Building (SMB), which is the tallest highrise in Japan. (Fig.5) The S.M.B., located in the New Shinjuku Business Center of Tokyo, is a highrise office building with 3 basement floors and 55 storied tower, whose total height above ground is 210 meters.

The structure of the building consists of steel framing for the tower part from the 2nd through 55th stories, steel and reinforced concrete composite framing for the 3rd basement through the first story. The foundation and other surrounding lower portions are constructed with ordinary reinforced concrete. The over-all dimensions of the structural system for a typical office floor are 58.4 meters long and 44.4 meters wide, as shown in Fig.6(A). The columns of longitudinal direction are spaced at distances of 3.2 meters, and these in the transverse direction are spaced at 15.6 meters for the office section, and 13.2 meters for the core portion. Those columns are built-up box section (500 x 500mm). Beams are built-up I section (800mm high) and by castellated beams.

The wind and earthquake resisting system consists of rigidly connected framings interacting with the slitted reinforced concrete shear-walls installed in the core portion, and large diagonally braced frames at both ends as shown in Fig.6(B).

SUMMARY

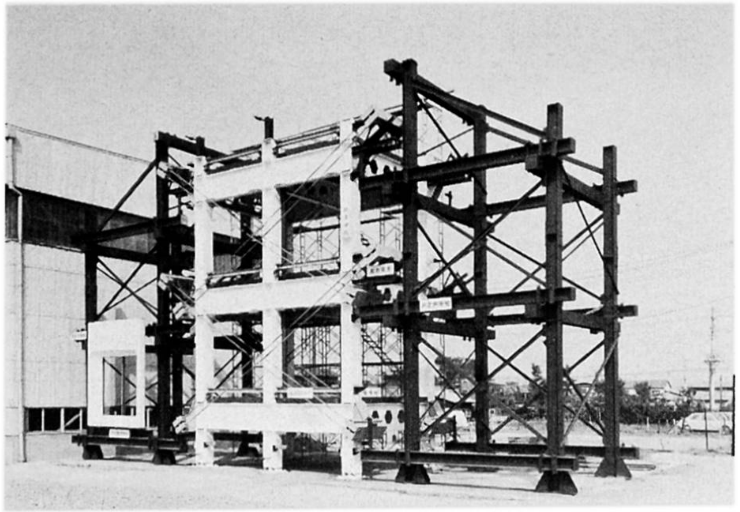
The feed-back dynamic design system has been accepted in Japan as the most advanced of its kind for tall buildings over 45 meters. Dynamic analysis using a rigorous computer program is indispensable in design procedure, as well as in earthquake observation and structural testing. By virtue of the major links in this system, we can control ultimate strength and deformation, quite apart from conventional safety factor concept.

RESUME

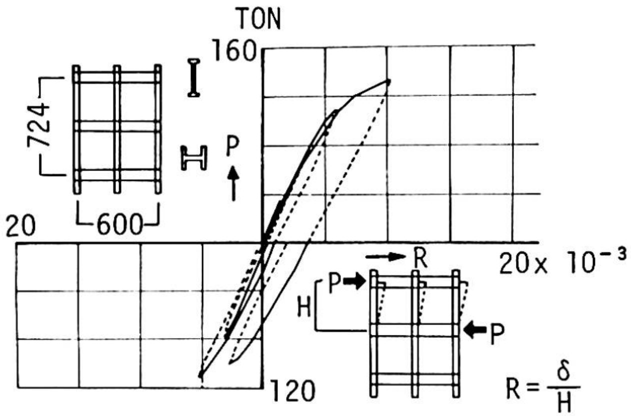
L'étude dynamique, par une méthode d'approximations successives, est reconnue comme la plus élaborée au Japon, pour les bâtiments de grande hauteur, de plus de 45 mètres. Une analyse dynamique, rigoureuse, par ordinateur, basée sur les observations sismiques et les résultats d'essais en laboratoires est indispensable. Lors du développement de chacune des boucles du programme, il est possible de contrôler l'état des contraintes limites et des déformations, indépendamment du facteur traditionnel de sécurité.

ZUSAMMENFASSUNG

Die iterative dynamische Berechnungsmethode wurde als die fortschrittlichste Methode bei Hochhäusern über 45 m in Japan angenommen. Die dynamische Untersuchung mit Hilfe eines genauen Computer-Programmes ist unerlässlich für das Berechnungsverfahren sowie für die Beobachtung des Erdbebens und für die Prüfung von Konstruktionen. Die Methode gestattet die Grenzfestigkeit und Verformung, unabhängig vom üblichen Sicherheitsfaktor-Denken, zu überprüfen.

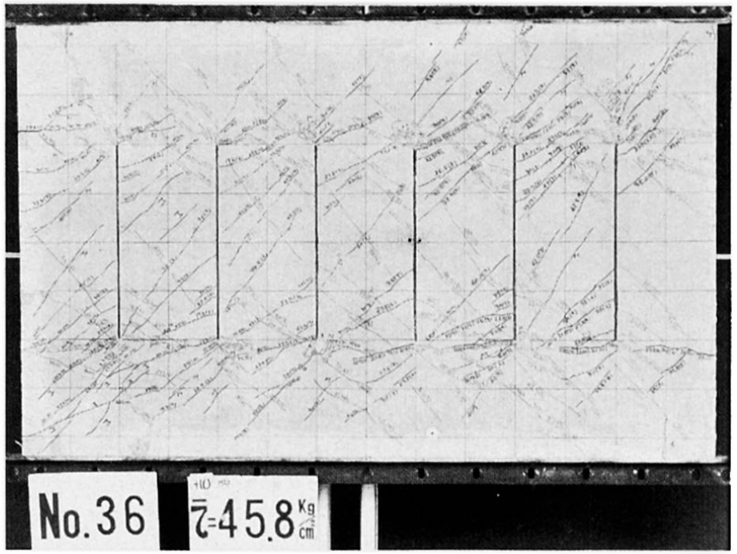


(a) General View of Full-Scale Test on Steel Frame

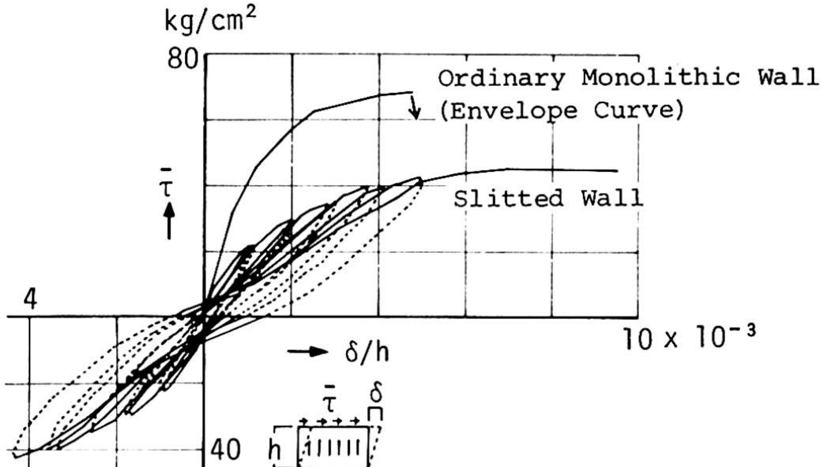


(b) Load-Deflection Relationship of Full-Scale Test

STEEL FRAME



(a) Crack pattern in a Slitted Test-Wall---When the applied shear force is 45.8kg/cm^2, the cracks are distributed finely with no majar diagonal cracks.



(b) Shear Stress vs. Sway Deflection Angle---The deformability of the slitted wall is more than twice that of an ordinary monolithic wall.

SLITTED WALL

FIG. 2 STRUCTURAL TESTS

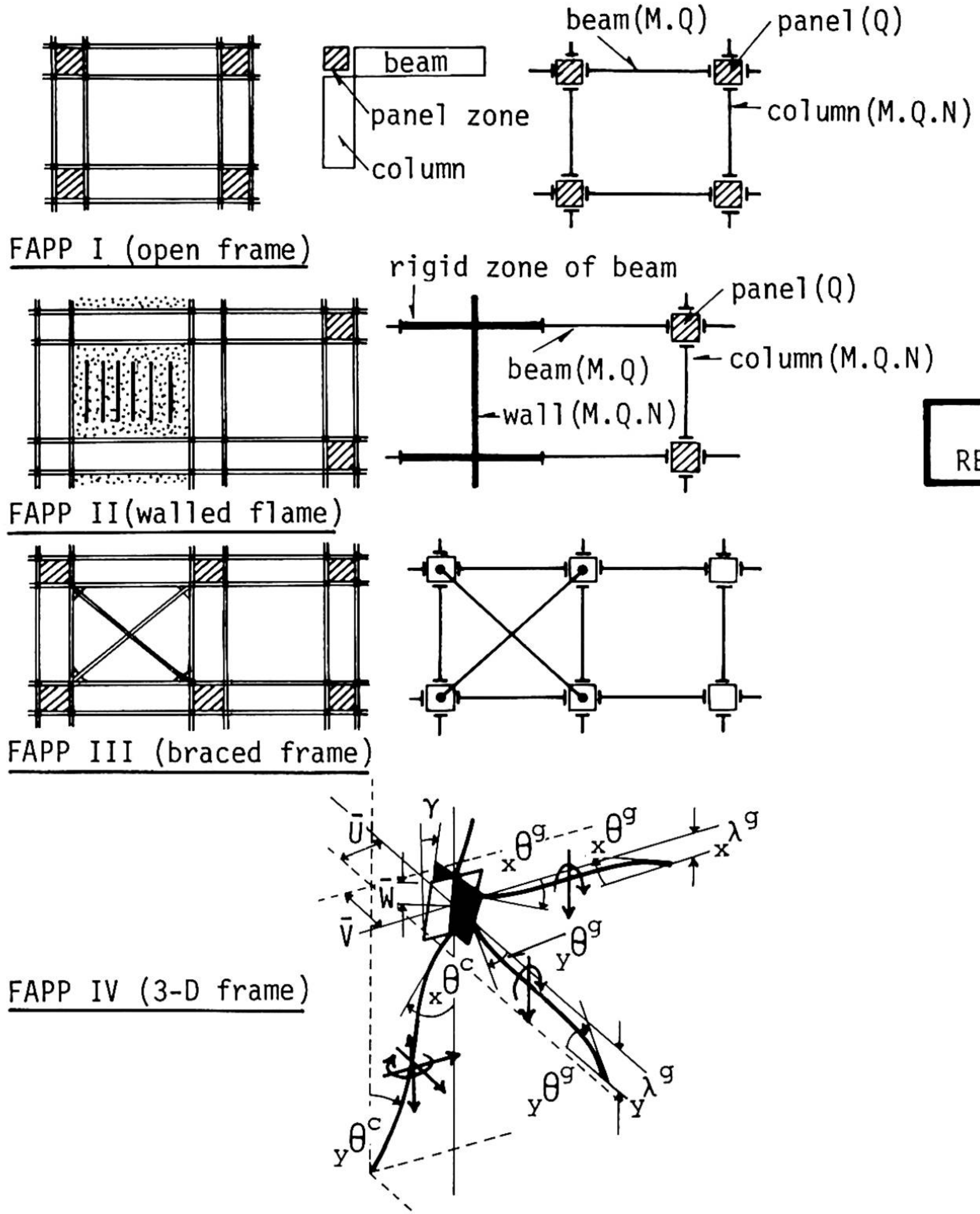


FIG.3 BASIC CONCEPT OF FAPP

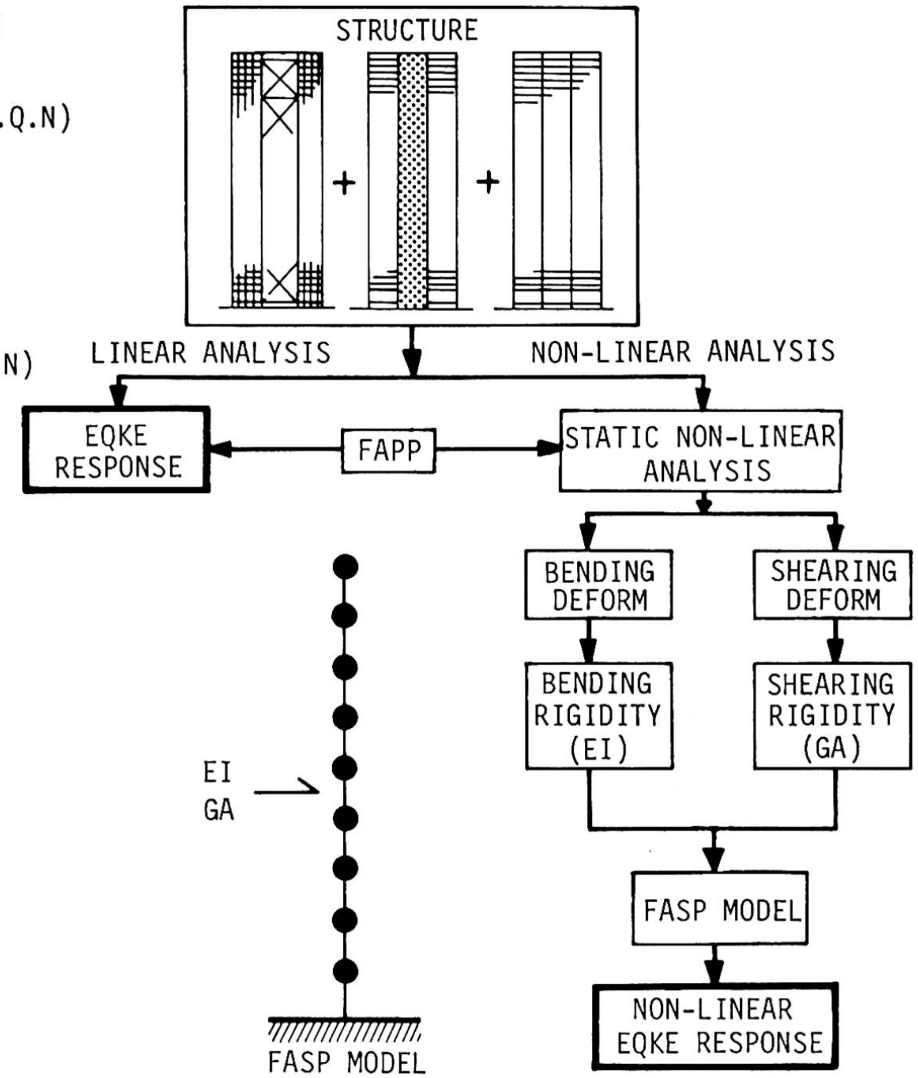


FIG.4 FAPP-FASP SYSTEM

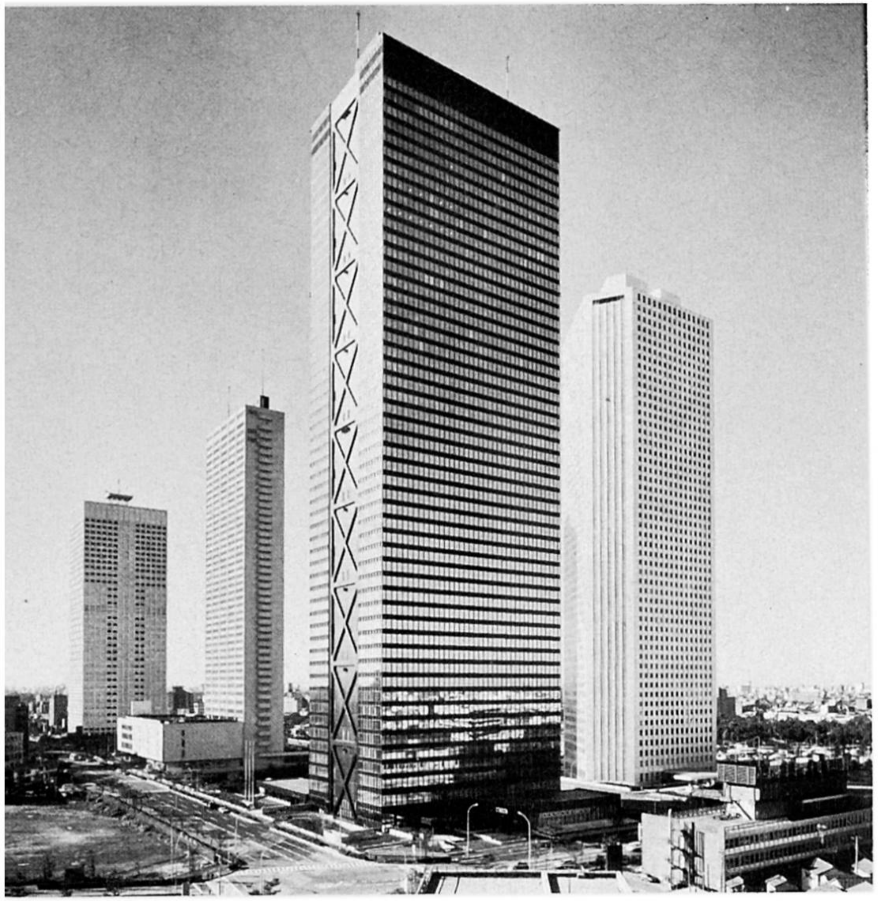
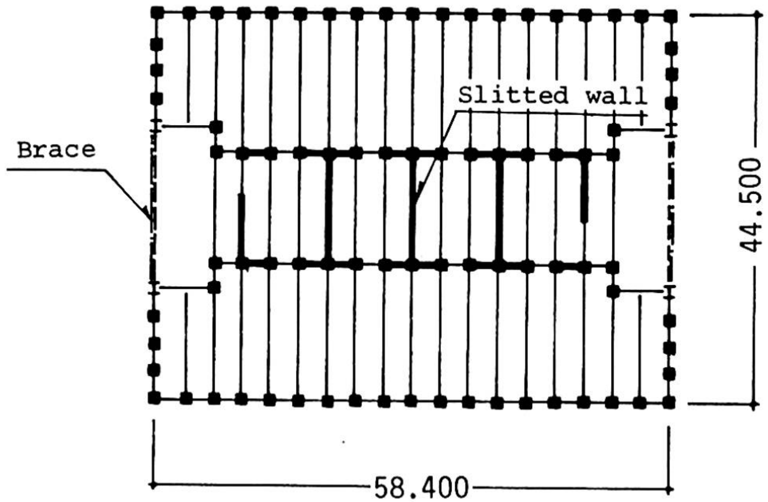
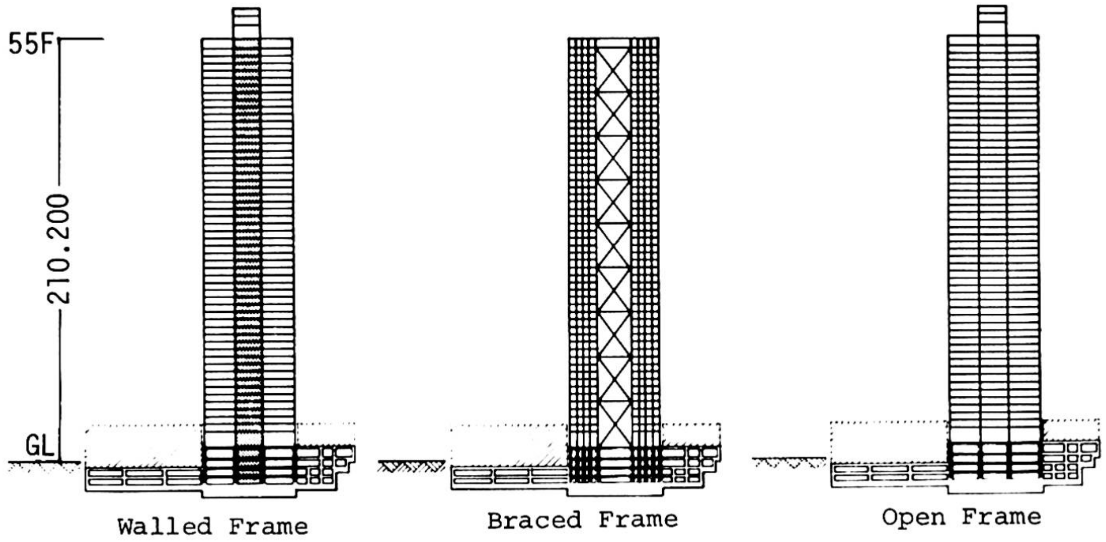


FIG. 5 SHINJUKU MITSUI BUILDING



(A) TYPICAL FLOOR PLAN



(B) EARTHQUAKE RESISTING FRAMES (TRANSVERSE)

FIG. 6 OUTLINE OF BUILDING STRUCTURE

Ic
La serviciabilité requise et l'entretien
Nutzung und Unterhalt
Serviceability and Maintenance

The James River Bridge.
Rescue Campaign for a 7 Kilometer Crossing

Le pont sur la rivière James.
Campagne de sauvetage d'un pont de 7 km

Die Brücke über den James River.
Rettungsaktion für eine 7 Kilometer lange Brücke

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To design a highway bridge crossing of a 7-km-wide estuary for economical construction and minimum maintenance cost is a technical challenge. To plan construction of such a facility without any source of funds introduces some practical problems. When the project involves replacement of a 50-year-old structure that is literally falling apart, piece by piece, the problems assume some urgency. The efforts to resolve these problems are best described not as a design, but as a campaign. This, then, is a recital of the campaign to rescue the James River Bridge.

The existing bridge was constructed in 1928 as a private toll facility. It consists of a 90-meter (m) vertical lift span, 730 m of steel truss and girder spans, and 6,200 m of 13.4-m steel stringer trestle spans. Since 1949 the bridge has been part of the Virginia State Highway System, forming one of the principal highway routes across the maze of estuaries of Tidewater Virginia (see Fig. 1). Traffic has grown to 4 million vehicles per year, with a peak of 1,000 per hour in one direction.

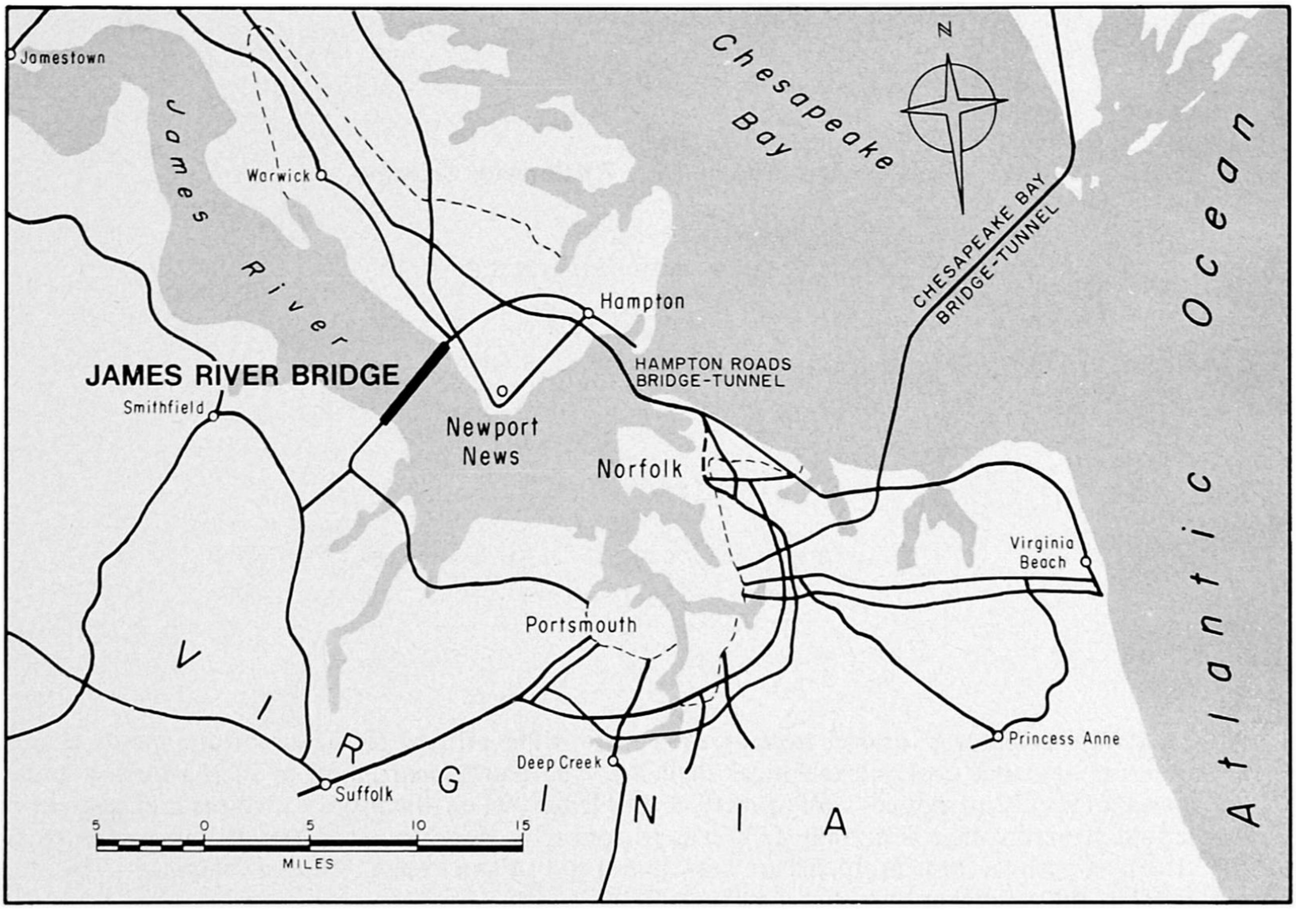
By the 1960s the 6.7-m-wide roadway had become seriously obsolescent, particularly with no facilities for handling breakdowns anywhere in the 7-km bridge length. Beyond this, physical deterioration of the structure was accelerating alarmingly, primarily from exposure to a salt-laden marine environment. Figures 2 and 3 show typical conditions on the trestle spans. There was serious concern that the bridge might actually collapse, and its replacement became a priority project of the Virginia Department of Highways.

Unfortunately, there was no money. All federal aid and state matching funds were committed to other projects of even higher priority. The $60 million cost of a new four-lane bridge was simply out of the question.

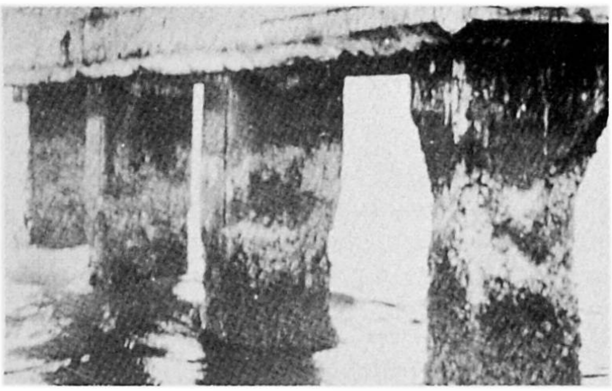
The only potential financing source was the Reserve Maintenance Fund of the Virginia Toll Bridge and Ferry System, to which the James River Bridge belongs. This could yield sufficient money for emergency repairs, and over a period of years might accumulate enough to start the reconstruction. But it was scheduled to go out of existence in 1976, when the completion of the federally financed Second Hampton Roads Bridge-Tunnel Crossing would trigger the removal of tolls throughout the system, redemption of bonds, and reversion of all reserve funds to the general State treasury. And in the most optimistic view, this fund might yield barely one-third of the money required for the new bridge.

Under these constraints, the rescue campaign evolved in four phases:

1. Repairs to hold the old bridge together while a construction fund was accumulated.



1. James River Bridge Location



2. Necked pile caps damaged by age and salt water

3. Deteriorated riding surface temporarily repaired with poured asphalt

2. Initial construction of a new 6.2-km two-lane trestle, with temporary crossover connections to the old 800-m channel crossing.

3. Addition of the four-lane channel crossing.

4. Addition of the second two-lane trestle to complete the full four-lane facility.

The timing of Phase 2 was critical – it had to be deferred until sufficient funds were accumulated but had to be completed before 1976 when the source of funds would disappear. The repairs of Phase 1 had to be sufficient to maintain the structure safely until its replacement parts were completed, but not so extensive as to deplete the funds being accumulated for reconstruction. And everything had to be done during a period of unpredictably escalating construction costs.

The key to the success of the campaign was the observation that 90 percent of the trouble with the old bridge was contained in the 6.2-km trestle, but only one-third of the reconstruction cost was. needed to replace this with a new two-lane structure.

The campaign thus developed with two objectives:

1. To manage and conserve the limited available resources to implement Phases 1 and 2.

2. To plan Phases 3 and 4 to minimize life cycle costs of the facility, including both capital construction costs and maintenance and operation costs..

The critical deterioration of the old trestle structure was concentrated in the concrete within the tidal zone and on the undersides of the pile caps and roadway deck. In these areas salt penetration had caused extensive spalling and softening of concrete and exposure and rusting of reinforcing steel. Many of the concrete piles were necked down alarmingly (see Fig. 2). Two thousand five hundred piles were repaired (after chipping away all deteriorated concrete) by encasing them in cylindrical concrete jackets placed within gasketed water-tight cofferdam forms. Eroded concrete on the underside of the deck, and the side and bottom surfaces of the pile bent caps, which had been most exposed to salt spray, was also chipped away and repaired with pneumatically projected mortar.

Fortunately, the structural steel was in relatively good condition, and it was decided that it could be maintained for the required duration with minimal isolated reinforcement, plus maintenance painting. Systematic repairs were necessary for the concrete disphragms joining the steel stringers at the ends of each span. Thirty percent of these diaphragms were cracked, and were reinforced with steel channels and brackets. The concrete deck surface was badly cracked, but it was decided to spend the available money on structural repair to the under-surface rather than cosmetic repair to the top surface. The cracks were, therefore, sealed with poured asphalt (see Fig. 3), and the crazy quilt patched riding surface served as a constant reminder that Phase 1 was a temporary program.

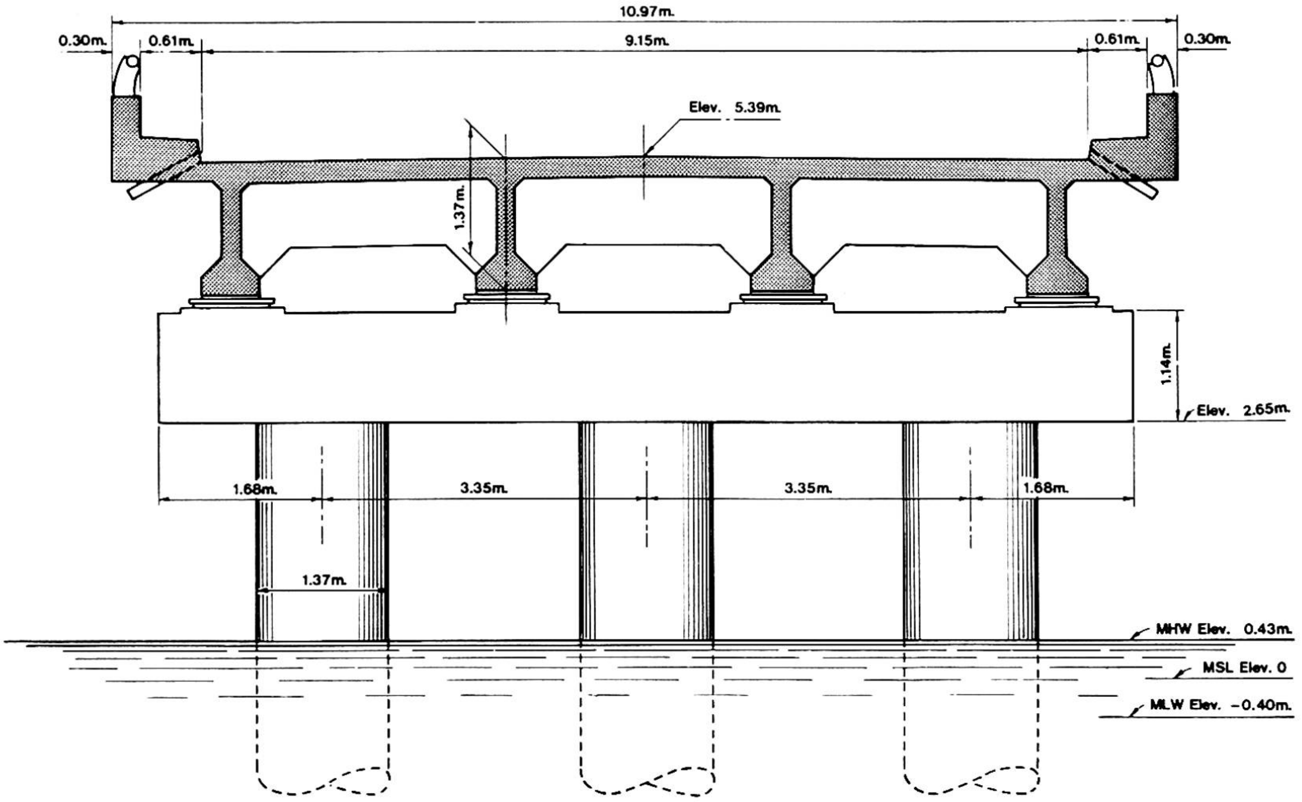
The design of Phase 2 had two objectives: to minimize initial construction costs of the new trestle structure and to eliminate the sources of deterioration that had been observed in the old bridge. Both objectives were served by designing a completely prefabricated structure of prestressed concrete, which minimized costly field labor on the exposed river site, and secured dense, crack-free concrete produced under factory-controlled conditions.

The cross section of the new trestle (see Fig. 4) features a 9.15-m-wide roadway with two 0.6-m safety walks and solid concrete parapets. The roadway affords sufficient width for two-way traffic past a stalled vehicle. The deck is composed of monolithic units 23 m long, consisting of four longitudinal beams, four transverse diaphragms, and a 20 cm deck slab. The entire unit weighed 230 tons.

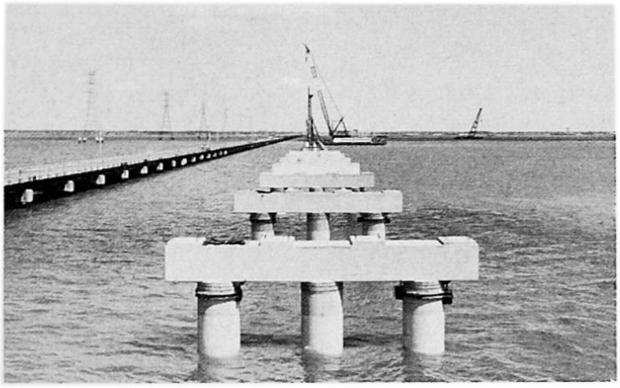
The safety walks and parapets were omitted from the precast monolith, and subsequently poured in situ. This eliminated any problems of aligning parapet and railing sections in adjacent spans, and facilitated installation of conduit and pullboxes for the roadway lighting system. Fortuitously, this also avoided a serious construction delay when a shortage of conduit developed.

Special design attention was given to eliminating creep deflection of the deck units to preclude "humping" of the spans over a prolonged service period, so that the roadway surface will remain smooth without requiring addition of a topping course. This was accomplished by arranging the prestressing tendons so that the unit stress under dead load plus prestress was uniform across the section at all points. Long-term creep will thus produce a uniform shortening of the span (estimated to be about 12 mm in 23 m), without bending deflection. The additional tendons necessary to accomplish uniform stressing added about 1½ percent to the cost of the trestle structure, compared to a design for strength only, without creep control.

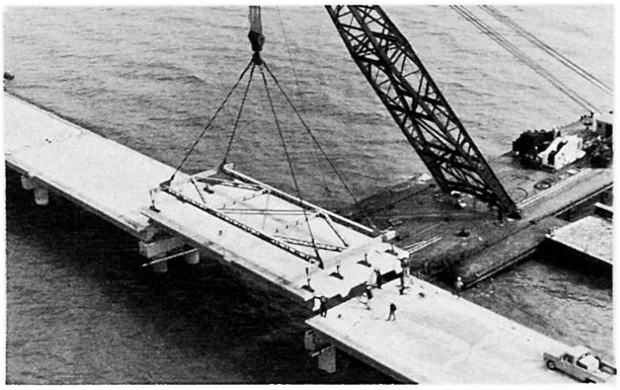
Open joints of 2.5 cm are provided across the full section of each pile bent support, to accommodate structural flexure of the simple span deck units, thermal expansion, and erection clearance for the installation of the precast monoliths. Short pipe sections through the deck slab at the gutter lines form scuppers to convey roadway drainage off the deck away from the open joints and pile bents.



4. Trestle cross section of new bridge design



5. Newly installed piles and caps for replacement roadway

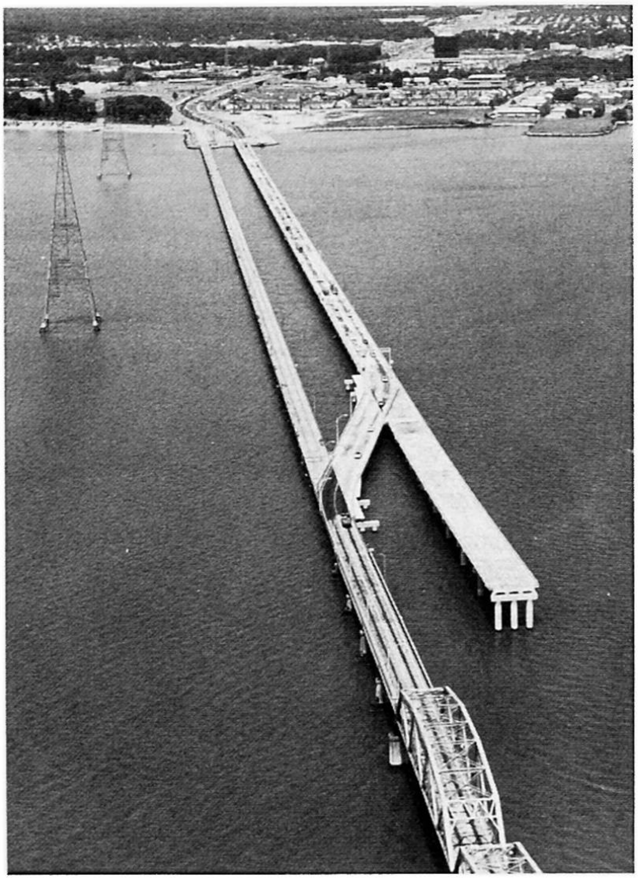


6. A span of new roadway being set in place

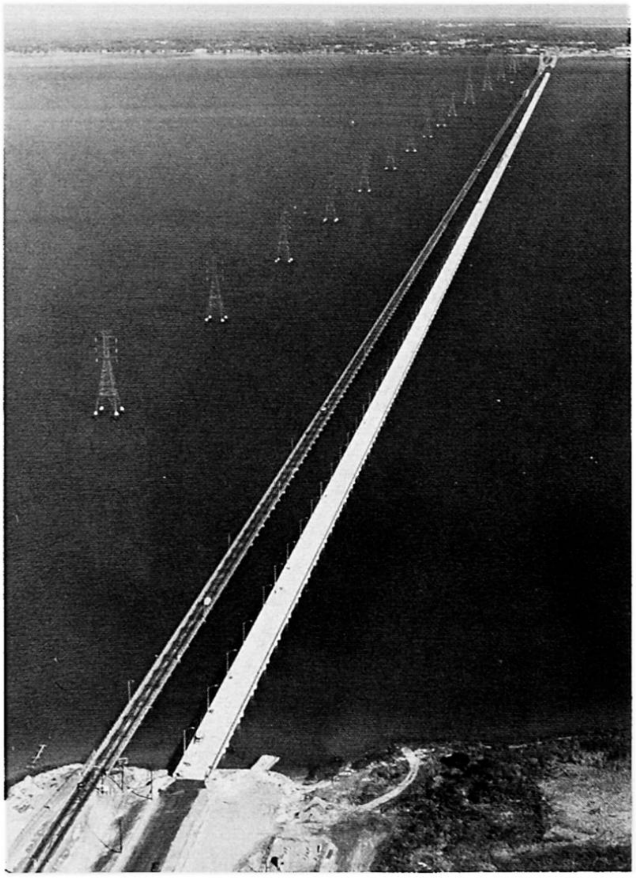
For the trestle foundations, precast, prestressed, cylindrical concrete piles were chosen. They have a diameter of 1.4 m and a wall thickness of 13 cm. Cast by a rotary centrifugal process, they have an exceptionally dense and smooth surface for resistance to salt water erosion. Over a 3-m height through the tidal zone they are further protected by an epoxy coating applied at the casting yard. The pile sections were precast in 5.2-m lengths, and joined by 16 prestressing cables to form individual piles up to 52 m long, weighing as much as 60 tons.

Three piles were connected by a precast, reinforced concrete cap to form a pile bent. The piles were filled with sand to furnish resistance against impact by small fishing vessels which abound in the area. A hollow void was left in the tops of the piles after they were cut off to a uniform elevation. The precast caps were fitted with three prongs of reinforcing steel that projected into the pile tops. After the caps were adjusted to grade and levelled, the voids were filled with concrete placed through vertical holes provided through the caps over each pile. The trestle substructure was thus established with a maximum propirtion of dense, high quality concrete and a minimum of site labor and materials.

The piles were set in prebored holes jetted into the alluvial river bottom, and then driven to a bearing stratum to develop a capacity of 180 tons each, in addition to the pile weight. Capacity was



7. *Old and new bridges shown during construction with crossover in operation*



8. *Aerial view of parallel crossings from south approach looking north*

confirmed by a series of 360-ton load tests. In granular soils, capacity was attained with only short driving. In silt and clay soils, it was necessary to drive the piles as much as 12 m. This required special precautions to avoid driving a plug of soft soil up inside the cylindrical pile, creating internal bursting pressures which could split the pile.

To minimize the effects of salt water spray on the deck units and pile caps, the crown of the new roadway was established at 5 m above mean high water (1.1 m higher than the old trestle). This places the bottom of the pile caps 2.2 m above mean high water. The selection of 3-pile bents was made to preserve stability in the event that any one pile was severely damaged by a collision.

Successful construction of the project is attributable to the availability and utilization of heavy marine equipment capable of handling and placing the 60-ton piles and 230-ton deck units within the close tolerances required to secure proper fit of the precast elements. The accuracy of final assembly is a tribute to the care and skill of the construction contractor, Tidewater-Raymond-Kiewit, and to the precaster, Bayshore Concrete Products Corporation.

The 6.2-km new trestle structure was completed and joined to the old channel crossing by temporary crossover structures in September 1975, at a cost of approximately $17 million, well within the available funds for Phase 2. This has permitted retiring the old trestle structure, and has eliminated 90 percent of the safety hazards of the crossing.

Design of Phase 3, the new channel crossing, has been completed, and construction is scheduled to start in the fall of 1976, when sufficient funds will become available. The old channel crossing consisted of a 90-m vertical lift span furnishing a 15-m vertical clearance in the down position. The new crossing is also a vertical lift, but the span has been increased to 126 m to provide a 106-m-wide navigation channel, and the vertical clearance was set at 18.3 m, which is estimated to reduce the required number of openings by over 50 percent.

A fixed, high-level crossing was considered as an alternative to the adopted design, but the required 44-m vertical clearance would have resulted in a long, high structure of expensive elements. In addition to an estimated $20 million extra capital expenditure for the complete four-lane project, this layout would have precluded the half-width construction of 1.5 km of the trestle approaches, and required maintaining a corresponding length of deteriorated old trestle in service until the completion of Phase 3. Finally, the energy cost of requiring all highway traffic to climb an additional 30 m was judged to be more objectionable than the relatively infrequent delays attributable to opening the movable span.

Phase 3 was designed as a full four-lane crossing for the main channel span and its foundations, with two-lane approaches connecting to the Phase 2 trestle structure. It was judged to be inadvisable from an operating viewpoint to have two separate, adjacent two-lane movable spans, and such a design would have involved a substantial cost premium for the overall project. Construction funds for a two-lane channel crossing could not have been obtained any sooner than those for a four-lane facility, so there was no scheduling advantage to splitting Phase 3.

Phase 3 is currently scheduled to be completed in 1979, permitting the full retirement of the old structure. Funding of Phase 4 is not yet scheduled, but on the presumption that this will be accomplished by 1980, the duration of the James River Bridge Campaign will have been 25 years.

One oddment remains. Despite fiscal and administrative constraints, the campaign to construct the new bridge has proceeded on a logical basis. Demolition of the old structure, however, promises to cause no end of controversy. The James River and the Hampton Roads estuary are prime fishing and shellfish grounds, and all schemes proposed to date for demolition and disposal of the old structure (particularly the solid masonry piers of the channel crossing) have aroused spirited environmental objections. It appears that it might become necessary to mount a new campaign.

SUMMARY

The campaign to replace a 50-year-old bridge without sufficient funds involved (1) temporary rehabilitation and maintenance, (2) construction of 6.2 km of two-lane trestle spans, (3) construction of 800-m, four-lane channel crossing, and (4) addition of parallel two-lane trestle. Design of the new structure includes a completely prefabricated trestle featuring dense crack-free concrete, epoxy protection for concrete piles, extra tendons to eliminate creep deflection, and 3.5-m underclearance to reduce salt spray erosion. Phases 1 and 2 have been completed, solving 90 percent of safety and maintenance problems with expenditure of one-third of total project funds.

RESUME

La campagne de reconstruction d'un pont cinquantenaire avec des fonds insuffisants a comporté: 1) Remise en état temporaire et entretien, 2) Construction d'un viaduc de 6.2 km, et à 2 voies, 3) Construction sur une longueur de 800 m d'une travée à 4 voies et 4) Addition d'un passage parallèle et à 2 voies. Le projet de la nouvelle structure prévoit des piles en béton, des câbles de précontrainte supplémentaires pour lutter contre le fluage et un tirant d'air de 3.5 m pour réduire l'érosion due au sel. Les phases 1 et 2 ont été exécutées, résolvant ainsi 90 % des problèmes de sécurité et d'entretien, et dépensant 1/3 des fonds du projet.

ZUSAMMENFASSUNG

Die Aktion, eine 50 Jahre alte Brücke ohne genügende Geldmittel zu ersetzen, erfolgte in mehreren Phasen: 1) Provisorische Wiederherstellung und Unterhalt, 2) Konstruktion eines zweispurigen, 6,2 km langen Viaduktes, 3) Konstruktion einer 800 m langen Brücke über den Schiffahrtskanal, und 4) Konstruktion eines zweiten parallelen Viaduktes. Die neue Brücke wurde als vollständig vorfabrizierter Viadukt entworfen, mit dichtem, rissfreiem Beton, Epoxy-Schutz für die Betonpfähle, zusätzlichen Vorspannkabeln zum Ausgleich der Kriechverformungen und mit 3,5 m Lichtraum zur Verringerung des Salzwasserangriffes. Die Phasen 1 und 2 sind fertiggestellt. Mit dem Aufwand eines Drittels der gesamten Baukosten wurden 90 % der Sicherheits- und Unterhalts-Probleme gelöst.