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ASSOCIATION INTERNATIONALE DES PONTS ET CHARPENTES
INTERNATIONALE VEREINIGUNG FÜR BRÜCKENBAU UND HOCHBAU
INTERNATIONAL ASSOCIATION FOR BRIDGE AND STRUCTURAL ENGINEERING

DIXIÈME CONGRÈS
ZEHNTER KONGRESS
TENTH CONGRESS

TOKYO
SEPTEMBER 6–11, 1976

RAPPORT FINAL
SCHLUSSBERICHT
FINAL REPORT

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HERAUSGEGEBEN VOM SEKRETARIAT DER IVBH IN ZÜRICH
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Préface

Pour qui sait observer et écouter, un séjour au Japon est une succession de surprises et une satisfaction extrême. En effet, ce pays qui ne possède pratiquement pas de matières premières essentielles est l'un des plus avancés sur le plan industriel. Ce pays qui est resté longtemps à l'écart des échanges internationaux est devenu l'un des plus ouverts. Mais, malgré l'irruption de la civilisation occidentale, on constate constamment une référence au passé, ce qui donne à la vie japonaise une qualité unique et impressionnante. Je suis persuadé que cet environnement particulier a été bénéfique pour notre 10ème congrès. Un grand nombre d'Occidentaux membres de notre Association se sont déplacés. Les ingénieurs japonais qui sillonnent le monde n'ont pas boudé une manifestation organisée chez eux, bien au contraire. La qualité des participants a permis des débats fructueux qui sont transcrits dans ce volume. Grâce à une organisation impeccable, nos séances se sont déroulées sans aucune anicroche, et si le mauvais temps a parfois causé des surprises désagréables, c'est pour mieux démontrer combien nos amis japonais sont habiles pour trouver les solutions de rechange ou pour minimiser les effets d'un incident grave.

Nos remerciements très sincères vont aux participants et tout particulièrement à ceux qui ont présenté des exposés de synthèse, de discussion préparée ou libre.

Nos remerciements les plus vifs vont au Comité d'organisation présidé par M. Yoshihiro Inayama. Lui-même et ses collègues ont bien mérité la reconnaissance de notre Association internationale des ponts et charpentes. Ma reconnaissance va également aux membres du Comité exécutif, de la Commission technique et des Commissions de travail. Grâce à leurs efforts et leur intelligence, notre congrès a été une grande rencontre qui a apporté incontestablement une pierre à la compréhension internationale, tant sur le plan scientifique et technique que sur le plan affectif.

Je donne rendez-vous à tous nos membres pour le 11ème congrès de l'AIPC en 1980 à Vienne.

Zurich, février 1977

Le Président de l'AIPC:

Prof. MAURICE COSANDEY
Président de l'Ecole Polytechnique Fédérale de Lausanne

Vorwort

Für denjenigen, der beobachten und zuhören kann, ist ein Aufenthalt in Japan eine einzigartige Folge von Überraschungen. Während langer Zeit im internationalen Austausch abseits stehend und arm an allen wesentlichen Rohstoffen, gehört Japan heute zu den offenstehendsten und in industrieller Hinsicht zu den fortgeschrittensten Ländern der Erde.

Trotz dieses Einbruchs westlicher Merkmale stellt der Besucher doch allenthalben enge Beziehungen zu alten Traditionen fest, welche dem japanischen Leben eine ganz einzigartige und beeindruckende Qualität verleihen. Ich bin überzeugt, dass diese besondere Umgebung unserem 10. Kongress förderlich war. Eine grosse Anzahl unserer Mitglieder aus westlichen Ländern hat sich auf die weite Reise gemacht und die welterfahrenen japanischen Ingenieure haben nicht gezögert, die in ihrem Lande angesetzte Veranstaltung zu einem Erfolg zu machen. Wissen und Fähigkeiten der Teilnehmer erlaubten fruchtbare Diskussionen, deren Ergebnisse im vorliegenden Band zusammengefasst sind. Dank einer in jeder Hinsicht einwandfreien Organisation konnten unsere Sitzungen ohne jede Schwierigkeit ablaufen. Wenn das schlechte Wetter auch manchmal unangenehme Überraschungen verursachte, so gab dies unseren japanischen Freunden nur umso bessere Gelegenheit, zu zeigen, wie gewandt sie sind, Ersatzlösungen zu finden oder die Folgen ernster Zwischenfälle klein zu halten.

Unser aufrichtiger Dank geht an alle Teilnehmer und ganz besonders an alle diejenigen, welche Beiträge zu den Veröffentlichungen eingereicht oder an der vorbereiteten oder der freien Diskussion teilgenommen haben.

Unser allerbester Dank gilt jedoch dem Organisationskomitee und dessen Vorsitzendem, Herrn Yoshihiro Inayama. Er und seine Kollegen verdienen die ausdrückliche Anerkennung unserer Internationalen Vereinigung für Brückenbau und Hochbau. Die Anerkennung gilt ebenfalls den Mitgliedern des Vorstandes, der Technischen Kommission und der Arbeitskommissionen. Dank ihren Anstrengungen war unser Kongress in wissenschaftlicher und technischer, aber auch in menschlicher Hinsicht ein unbestreitbarer Beitrag zur internationalen Verständigung.

Ich freue mich auf ein Wiedersehen mit allen unseren Mitgliedern am 11. Kongress der IVBH 1980 in Wien.

Zürich, Februar 1977

Der Präsident der IVBH:

Prof. MAURICE COSANDEY
Präsident der Eidgenössischen Technischen Hochschule Lausanne

Foreword

For those with the ability to observe and listen, a stay in Japan is a very rewarding experience which provides the visitor with a series of surprising facts. This country, which possesses practically no essential raw materials has now become one of the most advanced in the industrial world and has exchanged its rôle of bystander for that of one of the foremost nations in the international field. However, inspite of the intrusion of Western civilisation, much of the traditional form of living has been retained and it is this which makes the Japanese way of life so unique and impressive. I am sure it was this special environment which induced such a good attendance on the part of our Association's members from the West to our 10th Congress. Japanese engineers also, who are now so used to travelling all over the world, were also present in large numbers at this Meeting held under their own auspices. The high standard of knowledge and experience of those taking part produced very profitable discussions, reports of which are included in this volume.

Our meetings were organized to perfection and the bad weather which we sometimes experienced only served to demonstrate to us the aptitude of our Japanese friends for providing interesting alternatives or minimumizing unfortunate circumstances.

We wish here to thank very sincerely all those who attended the Congress and, most particularly, those who contributed Papers or took part in the Prepared or Free Discussions.

We should like to express our deepest gratitude to the Organizing Committee presided over by Mr. Yoshihiro Inayama. The IABSE has pleasure in taking this opportunity of acknowledging here the work which he and his colleagues so capably performed.

We extend also our grateful appreciation to the members of the Executive Committee, to the Technical Commissions and to the Working Commissions. Thanks to their able endeavours our Congress has made an indisputable contribution towards scientific, technical and personal understanding.

I look forward to meeting all of our members at the 11th Congress which will be held in 1980 in Vienna.

Zurich, February 1977

The President of IABSE:

Prof. MAURICE COSANDEY
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Allocution par le Président du Conseil Japonais de la Science

Ansprache des Vorsitzenden des wissenschaftlichen Rats von Japan

YUICHI OCHI
Chairman
Science Council of Japan
Japan

Mr. President, Ladies and Gentlemen:

At the opening of the 10th Congress of the International Association for Bridge and Structural Engineering, I feel highly honored to be given an opportunity of extending my hearty congratulations on behalf of the host organizations.

First of all, I would like to offer a cordial welcome to the directors and committee members of the International Association for Bridge and Structural Engineering, and also to the scientists having come over to Japan from distant parts of the world to attend this congress.

It is needless to mention the remarkable progress which structural engineering has recently attained in the technical application as well as the fundamental studies. Every participants in this congress is expected to exchange his constructive opinions positively with each other and possibly in a friendly manner. I believe thus the scientific progress in this particular field will properly be accelerated.

The Science Council of Japan has devoted its efforts to the fullest capacity in advancing sciences and is well aware of the importance of international exchanges of scientific knowledge. You are heartily requested to make the most of this opportunity towards promoting mutual cooperation among nations and thus to ascertain a beneficial significance of this congress.

Last but not least, I sincerely hope that you will all come to acknowledge this congress as a brilliant success.

Thank you.

Welcome Speech

Discours de bienvenue

Willkommensansprache

YOSHIHIRO INAYAMA
Chairman
Japanese Organizing Committee
Japan

Mr. President Cosandey, Mr. Takeshita, Minister of Construction of Japan, Ladies and Gentlemen:

On behalf of the Japanese Organizing Committee of the 10th Congress of the International Association for Bridge and Structural Engineering, I wish to extend a most cordial welcome to you. As chairman of the committee, it gives me great pleasure to note the presence at this inaugural session of the large number of those who have come from many different parts of the world.

We started preparations for this congress three years ago, in August. A few months later, the world entered a period of profound changes caused by the oil crisis. This gave rise to serious difficulties for the association's headquarters in Zurich as well as for all those connected with the plans for this congress, including our organizing committee.

However, I wish to join you all in rejoicing that thanks to the efforts and firm determination of all concerned we are able today to hold this grand opening session.

I wish to express my profound thanks to Professor Maurice Cosandey, president of our association, and to Mr. Takeo Atsumi, chairman of the Japanese Fund Raising Committee and others, who through their efforts have made this possible.

Needless to say, the prosperity of the modern society depends upon many developments of science and technology. As we look back on the past, history of mankind of some two thousand years can be called as the history of wars. It is true that wars brought about great technological development, sometimes drastically and markedly, and sometimes step by step. However, it was with many sacrifices and destructions. The world now is greatly changing of late because of global efforts to attain peace. Of course peace has not yet been attained throughout our global society but it cannot be denied that we all are making best efforts toward building a peaceful world. And I believe it is not a mere dream.

This peaceful world, attained after two thousand years, will depend on the co-existence and common prosperity of all humanity. It will call for cooperation, not for

war, fighting, or desperate competition in which people hurt one another. Cooperation and coordination is the key here. So we are now standing in front of the world which is trying to obtain the peaceful coexistence and cooperation.

I believe that in the field of structural engineering, as in all other fields, there will continue to be increasing demands that progress in science and technology must further human prosperity and happiness. Such demands call for increased mutual understanding and cooperation, as I have already mentioned, among all members of our association.

Colleagues, is it not true that there is no other way than for all to join hands for the advancement of human prosperity, with each country benefitting from the strong points of others and helping make good the shortcomings of others?

It is thus that I look forward with great expectations to the fruitful results of this congress.

In addition to the technical sessions, our committee has included various other events in the congress program. I hope that all of you who have come all the way here from many parts of the world, will enjoy your stay in Japan and have an unhurried opportunity to observe the varied aspects of our country--so old and yet so new.

This oriental nation has, within the framework of world history, a quite distinctive feature. Our culture, of which we are so proud, may appear strange to you. But this culture has been nurtured by our forebears over a period of a thousand and more years, and I am convinced that you will find it of a distinctive flavor capable of deep appreciation upon close and unhurried observation.

And at the same time we also have the "modern" aspect of Japan you see today which was built up by introducing no-legend culture from many other parts of the world in the past one hundred years since this country launched itself on the path of rapid modernization. It is my wish that, without fail, you will see different aspects of our country.

In conclusion, I wish to express my profound thanks to our distinguished guests, the president and members of our association and to the participants from various countries for their cooperation in making this congress possible.

Thank you.

Address by the Minister of Construction

Allocution par le Ministre de la Construction

Ansprache des Bauministers

NOBORU TAKESHITA
Minister of Construction
Japan

Your Excellencies, Distinguished Guests, Ladies and Gentlemen:

It is a great honor for me to have been accorded this opportunity, in my capacity as Minister of Construction of the host country, to address the participants from numerous countries at the 10th Congress of the International Association for Bridge and Structural Engineering.

The contributions made by your association over the last 50 years toward the progress and development of the science of structural engineering merit the greatest commendation. It is a particular pleasure for me, as a representative in the administration of construction activities, to note that this congress, held every four years, should have chosen Japan for its 10th meeting.

As you are well aware, our country supports a large population on narrow, mountainous territory, often afflicted by typhoons and earthquakes.

In spite of these unfavorable natural conditions, we have succeeded in overcoming various obstacles in the construction of high-rise buildings, long-span bridges and other structures.

In order to achieve effective use of our limited land area and, also, in order to improve the social welfare of our people, we are faced with the need to further pursue our construction efforts.

To accomplish this, we look forward to the further progress in basic technologies and, in particular, in research and development in the field of structural engineering.

I deem it, therefore, highly significant that this congress, bringing together authorities and experts in this field, have been convened here today. I wish to extend to all of you the warmest welcome and, at the same time, to express my profound interest in the results of this meeting.

I have been informed that in addition to the technical sessions, a varied program of study tours and entertainment has been arranged so that you may better understand Japan. It is my sincere hope that you will not only learn much about present conditions

in our country, but will also reap a rich harvest of pleasant memories.

In closing, I wish to express my deep respect to the president of your association and to the president of the Japanese Organizing Committee for this Congress and to their associates for their untiring efforts to bring about this meeting, which I fervently hope will be most successful and fruitful. I also wish your association continued success and development.

Thank you.

Discours inaugural

Eröffnungsrede

Inaugural Speech

MAURICE COSANDEY
Professeur
Président de l'AIPC
Lausanne, Suisse

Le Japon, dont le nom et l'emblème rappellent le soleil, source de chaleur et de lumière, est un peuple et un pays dont l'histoire et le rayonnement sont à la mesure de la destinée de l'humanité, laquelle malgré les énormes contradictions actuelles peut être considérée comme exhaltante et grandiose. Aussi je suis heureux de déclarer que l'invitation de nos amis japonais de tenir notre 10ème congrès à Tokyo est un honneur qui a été ressenti avec émotion, enthousiasme et reconnaissance. C'est pour nous aussi l'occasion de reconnaître le génie créateur propre des ingénieurs, architectes et entrepreneurs japonais, et notamment de ceux qui font partie de notre Association dont nous avons toujours apprécié la haute compétence et l'originalité. De nombreux domaines peuvent attester la valeur de mon propos. Qu'il suffise de penser, comme cas particuliers, aux constructions hautes asismiques et aux infrastructures ferroviaires.

L'organisation d'un congrès, tous les quatre ans, de l'Association internationale des ponts et charpentes est une opération difficile en raison de son importance et de son niveau. Le comité d'organisation japonais l'a magistralement réalisée, et c'est avec une profonde admiration que nous remercions tous les membres de ce comité ayant à sa tête Monsieur Yoshihino Inayama, Président, et Monsieur le Professeur Takeo Naka, Vice-Président. Qu'il me soit permis également d'associer à cet hommage toutes les personnes et sociétés, lesquelles, de près ou de loin, ont apporté leur concours moral ou financier, ou les deux, à ce grand événement que représente notre rencontre 1976 à Tokyo.

La nation, l'entreprise, l'association ou l'individu basent leur activité sur trois grands piliers ou composantes: la connaissance, l'action et la foi, ce dernier mot devant être pris dans sa signification la plus large à contenu profane et spirituel. La contribution de la nation japonaise à l'accroissement des connaissances est considérable et notre rencontre ici sera pour beaucoup d'entre nous l'occasion d'en faire, une fois de plus, la constatation. Dans le domaine de l'action, la démonstration n'est plus à faire de l'efficacité de celle du peuple japonais. Quant à la foi, je pourrais donner de multiples exemples. Mais je me limiterai à un seul: l'importance attachée au savoir et au progrès par l'éducation et la culture, et qui se traduit par une motivation intense et une curiosité intelligente. Bien sûr, comme chacun d'entre nous, le Japon pré-

sente des ombres, mais la conjonction des trois composantes citées plus haut me paraît meilleure que partout ailleurs, ce qui doit contribuer à faire de notre séjour ici l'un des moments privilégiés de notre vie.

Notre 10ème congrès aborde des thèmes fort importants de l'art de l'ingénieur, de l'architecte ou de l'entrepreneur. Une fois de plus, nous constaterons que le profit retiré dépend beaucoup de l'attitude générale adoptée. Plus elle sera ouverte et large, meilleure sera la compréhension des sujets particuliers. Il y a là une analogie avec l'exercice lui-même de la profession d'ingénieur. En effet, ce dernier doit se livrer, au début d'un processus, à toute une série d'analyses. Il n'atteindrait cependant pas son objectif sans une concrétisation de ses idées sous la forme d'une structure ou d'une machine dont le fonctionnement est vérifié et garanti. Mais la synthèse ne peut se limiter à la formalisation. Il y a lieu de faire intervenir les aspects sociaux-économiques qui englobent la considération des besoins de l'homme. L'esprit scientifique et technique n'est donc pas le seul à intervenir dans l'édification d'un ouvrage, mais il faut aussi mettre en oeuvre la faculté de saisir les interrelations plus larges qu'implique l'insertion d'un corps somme toute étranger dans un environnement donné. Ainsi, la construction d'une route modifie sensiblement le devenir de la région impliquée. En disant cela, il est évident que cette manière large d'aborder un problème comporte ses propres limites. Il n'est pas possible de concevoir une nouvelle société chaque fois qu'un projet d'une certaine envergure est mis en route. Mon propos est plus modeste. Il veut attirer l'attention sur le fait que l'activité de l'ingénieur a pris aujourd'hui une ampleur qui dépasse la simple collaboration avec le maître de l'ouvrage, le planificateur, l'architecte et l'entrepreneur. Il exige une relation avec la culture dans sa signification la plus large. Et je rejoins ici mes mots de tout à l'heure au sujet de l'attitude à observer au sein d'une rencontre comme la nôtre. Ecouter et faire l'effort de comprendre fait partie de la culture.

Notre association se distingue d'autres plus spécialisées précisément par ses buts de traiter les problèmes de structures dans leur contenu le plus large. Comme, par ailleurs, il est toujours inapproprié de se disperser, notre champ d'activité est divisé en cinq parties dont la responsabilité incombe à des commissions multinationales. La première s'occupe des questions générales et soutient l'action prospective. Son rôle est d'ouvrir de nouvelles voies en recherchant des concepts valables si possible pour tous les pays et tous les matériaux. Cette action se fait en collaboration éventuelle avec d'autres associations, mais la double qualité de généraliste et de spécialiste de nos membres prédestine notre association à un rôle majeur. Deux commissions sont consacrées aux problèmes plus spécifiques des structures en béton, béton armé et béton précontraint et à celles en acier ou mixtes. Elles représentent plus spécifiquement les préoccupations de l'ingénieur, lequel ayant conçu son oeuvre doit faire appel aux théories les plus affinées pour la calculer et aux meilleures expériences pour la dessiner. Mais le travail intellectuel ne suffit pas à garantir la qualité d'un ouvrage. Les problèmes de fabrication, en usine ou sur le site, sont tout aussi importants et peuvent même être déterminants quant à la sécurité et à l'économie. Et je voudrais là rompre une lance en faveur de l'action globale de l'ingénieur et de l'architecte. La créativité qui est l'un des apanages de l'humanité n'est pas limitée à la recherche ou au développement, mais elle se rencontre et elle est nécessaire à toutes les étapes aboutissant à l'objet fini, que celui-ci soit une maison, un tableau ou une poterie. C'est en méconnaissant ce principe que l'on a dissocié parfois recherche et enseignement au sein des universités. Certes, cette disso-

ciation est parfois nécessaire lorsque les impératifs économiques et les délais l'emportent sur l'optimalisation globale, mais elle n'est pas à recommander. C'est pourquoi notre association comprend en qualité de membres des chercheurs et des praticiens. Pour accentuer encore cette volonté de liaison entre la théorie et la pratique, entre la science et l'intuition, nous avons mis en oeuvre deux commissions orientées l'une vers les problèmes de réalisation, et notamment leur aspect économique, et l'autre vers la globalité de l'art de bâtir, et plus particulièrement vers la synthèse de la planification, du projet et de la réalisation. Je pourrais ajouter du bouclement des comptes. Mais ce dernier problème n'existe pratiquement pas si les premiers cités sont exécutés avec le soin et l'esprit voulus. Grâce aux travaux qui découleront des réflexions de ces nouvelles commissions, nous espérons contribuer à un nouvel essor des ponts et charpentes intégrés dans un univers plus vaste, guidé lui-même grâce à l'outil fascinant que représente l'analyse des systèmes.

Mais vous savez comme moi que la machine la plus perfectionnée, la théorie la plus sophistiquée ne peut résoudre aucun des problèmes vitaux de notre société. L'homme ne retire d'une calculatrice ultra rapide et ultra puissante que ce qu'il y a mis dans la programmation. S'il n'y a pas mis une partie de son coeur, il ne trouvera à la sortie que formules et signes cabalistiques. Ceci, bien sûr, n'est qu'un symbole, mais il n'est pas malséant de le rappeler à l'occasion. En effet, l'égoïsme personnel contrecarre la générosité et oublie que, souvent, en donnant satisfaction à l'intérêt général, on retrouve ultérieurement en bénéfice multiplié le sacrifice initialement consenti. Mais me voilà prisonnier de mon propre raisonnement. En effet, peut-on faire quelque chose sur cette planète sans en escompter un bénéfice ? Ma conviction est faite et ma réponse est oui. En voulez-vous une preuve ? Notre association internationale des ponts et charpentes est ouverte à tous les ingénieurs et architectes du monde. Cela constitue un potentiel d'échanges, de dialogues et de solidarité considérable. Au cours de nos rencontres, des informations sont échangées avec le seul intérêt de sentir une commune passion pour un but partagé: construire. Le sentiment d'appartenance à une communauté scientifique et technique renforce singulièrement le désir de voir l'humanité trouver la voie d'une vraie solidarité. Que nos amitiés mutuelles nées en divers points du globe contribuent à faire de cette réflexion idéaliste une réalité.

En déclarant ouvert le 10ème congrès de l'Association internationale des ponts et charpentes, je forme les voeux les plus chaleureux pour Sa Majesté l'Empereur du Japon, pour sa famille et pour le peuple japonais.

I

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

Ia

L'influence des moyens et des méthodes de construction

Einfluss der Baumethoden auf den Entwurf von Tragwerken

Planning of Structures and its Relationship with Construction Methods

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

Ic

La serviciabilité requise et l'entretien

Nutzung und Unterhalt

Serviceability and Maintenance

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Bemerkungen des Verfassers des Einführungsberichtes

Comments by the Author of the Introductory Report

Remarques de l'auteur du rapport introductif

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Die gegenseitige Abhängigkeit von Entwurf, Ausführung und Nutzung bei der Gestaltung von Tragwerken.

1. ZUR PROBLEMSTELLUNG

Im Einführungsbericht wurden die Hauptprobleme aufgearbeitet, die sich bei der Betrachtung des Einflusses der Baumethoden auf den Entwurf von Tragwerken stellen.

Das Entwerfen und Ausführen eines Tragwerkes ist eine Teilaufgabe, die dann sinnvoll gelöst werden kann, wenn sie im Rahmen der Gesamtaufgabe verstanden und bearbeitet wird.

Der Bauprozess ist ein äusserst komplexer Vorgang; wenn wir wesentliche Fortschritte erzielen wollen, müssen wir mehr über diesen Prozess wissen, Ursache-Wirkung-Beziehungen ergründen und in theoretischen Ansätzen fassen. Wir verfügen heute in Bezug auf die Ausführung von Bauten über einen guten "Know-how", es fehlen uns aber passende Methoden für deren Planung.

Die wesentlichsten Entscheidungen in einem Bauprozess werden im Frühstadium bei relativ tiefem Informationsstand gefällt. Umsomehr sind wir auf Methoden angewiesen, mit denen die wichtigsten Lösungsalternativen nicht nur gefunden, sondern vor allem bewertet werden können.

Mit dem Einführungsbericht sollten Diskussionsbeiträge provoziert werden, die am praktischen Beispiel die Entwicklung von Entscheidungskriterien im Zusammenhang mit der Bewertung von alternativen Tragwerksentwürfen zur Darstellung bringen würden. Der Einfluss der Baumethode wäre dann als ein Aspekt zur Darstellung gekommen.

Die Beiträge beschränkten sich leider auf die Beschreibung des ausgeführten Entwurfes unter besonderer Berücksichtigung des Zusammenhanges zwischen Baumethode und Entwurf. Das Problem Wählen unter verschiedenen Entwürfen kam nicht zur Darstellung. Der Grund dafür wird wohl in der enormen Arbeit liegen, die mit der Aufarbeitung dieses Problems verbunden wäre, dabei findet mangels passender Methoden dieser Wahlprozess eben oft unbewusst und nicht systematisch statt. Er ist dann eben auch nicht nachvollziehbar. Lernen wird man aber nur über ausgewertete Erfahrung. Wesentliche Fortschritte werden wir dann wieder erzielen, wenn wir nicht nur auf der technischen Seite hervorragende Lösungen erarbeiten, sondern das technische Produkt auch tatsächlich den langfristigen Bedürfnissen entspricht

Vor allem werden wir vermehrt das Bauwerk als Ganzes und für die ganze Lebensdauer betrachten müssen, denn wir werden zunehmend ja auch für die Umwelt verantwortlich gemacht.

2. GLIEDERUNG DER BEITRAEGE ZU THEMA Ia

Die Bedeutung der Teilaufgabe "Tragwerk" innerhalb eines Bauprozesses ist je nach Bauwerksgruppe sehr verschieden. Handelt es sich um die Gruppe der Brückenbauten, Turmbauten, Staumauern, Off-Shore-Tragwerke, wird die Teilaufgabe "Tragwerk" praktisch zur Hauptaufgabe. In dieser Gruppe ist die Hauptfunktion des Bauwerkes eine Tragwerksfunktion. Bei der Gruppe der Industriebauten, Verwaltungsbauten, Wohnbauten stellt die Aufgabe "Tragwerk" eine der wichtigeren Teilaufgaben dar. Es gibt hier keine Teilaufgabe mehr, die den Rang der Hauptaufgabe übernehmen könnte und nach der sich dann alle andern Aufgaben praktisch zu richten hätten. In der Gruppe Kanalbauten, Leistungsbauten, Pistenbauten, Strassenbauten spielt die Aufgabe "Tragwerk" eine untergeordnete Rolle.

Auf dem Hintergrund der vorstehenden Ueberlegungen kann man das Problem des Einflusses der Baumethoden auf den Entwurf von Tragwerken leicht einordnen. Einen Einfluss haben baumethodische Aspekte auf den Entwurf des Tragwerkes dann, wenn die entsprechenden Kriterien mit einem starken Gewicht in die Bewertung der Alternativen eingehen. Für Bauwerke der Gruppe A wird das meistens der Fall sein. Bei Bauwerken der Gruppe B werden bei den baumethodischen Betrachtungen nicht nur die Aufgabe "Tragwerk", sondern auch mehrere andere Aufgaben wie "Installationen", "Aussenhaut" etc. eine Rolle spielen. Bei den Bauwerken der Gruppe C stehen wohl die Baumethoden im Vordergrund, aber da das Tragwerk eine untergeordnete Rolle spielt, ist der Zusammenhang auf Stufe Bauwerk und nicht Tragwerk zu suchen. Im Bild sind Zusammenhänge dargestellt und die Beiträge eingeordnet.

	GRUPPE A	GRUPPE B	GRUPPE C
	BRUECKENBAUTEN TURMBAUTEN STAUMAUERN OFF-SHORE-BAUTEN	INDUSTRIEBAUTEN VERWALTUNGSBAUTEN WOHNBAUTEN	KANALBAUTEN PISTENBAUTEN LEITUNGSBAUTEN STRASSENBAUTEN
STELLUNG TRAGWERK ZU BAUWERK	HAUPTAUFGABE	TEILAUFGABE	NEBENAUFGABE
ZUSAMMENHANG BAUMETHODEN MIT ENTWURF TRAGWERK	GROSS	MITTEL	KLEIN
BEITRAG: AHORNER-JOHN BEITRAG: SCHLAICH-MAYR	UN-TOWER WIEN SEILNETZ-KUEHLTURM		
BEITRAG: MASON BEITRAG: MIYAZAKI BEITRAG: NARUSE-OIKE	CITY-HOCHSTRASSEN BOGENBRUECKE GROSSEL.-BRUECKEN		
BEITRAG: LEFEVRE-MAHIEU	STANDARDBRUECKEN		

Bild: Einordnung der Beiträge in die Problemstellung zum Thema Ia

In den sechs Beiträgen zu diesem Teilthema werden Bauwerke aus der Gruppe A und der Zusammenhang zwischen Baumethode und Entwurf beschrieben. Insbesondere zeigen die Herren Ahornen und John aus Oesterreich am Beispiel der UN-Towers in Wien und die Herren Schlaich und Mayr aus der BRD an einem Kühlturm mit Seilnetzmantel, dass die Wahl der Baumethode zum Entwurf des Tragwerkes gehören soll. Herr Mason demonstriert am Beispiel von City-Hochstrassen, Herr Miyazaki an einem Bogenbrückenbau ohne Lehrgerüst, die Herren Naruse und Oike beim dezentralisierten blockweisen Bau von Brückenabschnitten mit nachfolgendem Zusammenfügen wie stark gerade im Brückenbau die Baumethode den Entwurf des Tragwerkes beeinflussen kann. Die Herren Lefevre und Mahieu zeigen, dass eine Standardisierung im Brückenbau den Entwurf und die Baumethode umfassen muss.

In der freien Diskussion wurde ebenfalls zu den Hauptproblemen nicht Stellung genommen. Den Verfassern der Hauptbeiträge und den Teilnehmern an der freien Diskussion möchte ich für die Mitarbeit danken.

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Comments by the Author of the Introductory Report

Remarques de l'auteur du rapport introductif

Bemerkungen des Verfassers des Einführungsberichtes

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Achievement of Safety and Economy in Design and Construction

1. The relative character of safety and economy

In everyday usage the word "safe" means "no danger". Since we are here concerned with safety in relation to buildings and other structures, it is linked to the concept of "protectedness". The protectedness of the womb has its continuation in a need for protection from dangers. Those dangers threaten us out of doors; indoors we want to be safe. This is so deep-rooted a feeling that the emotional value of safety is absolute. We do not want to know about any risk of insecurity, any danger, except as an Act of God or a blow delivered by fate. If insecurity does not arise from either of these causes, but is instead due to some form of human action, we feel that it ought to be punished.

We are more willing to accept insecurity, an increased lack of safety, during the construction of structures. As regards the aspect of material damage or loss, the consequences of insecurity can be put on a level with other forms of damage and thus be regarded as a purely economic problem.

From the psychological point of view, however, insecurity goes far beyond this. Even if the consequences are confined to material damage, there remains the sense of insecurity in circumstances where we demand protectedness. For the rest, we are very inconsistent; we risk our lives

in busy traffic, we go on hazardous journeys, we participate in risky sports activities, we smoke too much, eat too much, have too much to drink before driving our cars. We are prepared to take risks, but not within our man-made environment itself.

If we so designed our buildings that every factor capable of presenting a hazard, causing insecurity, were taken care of, so that we could say "we have thought of everything; now we are safe", they would not only be impossibly expensive, but also uninhabitable in consequence of the fear embodied in every feature of such buildings. And, anyway, we should even then have to admit that the absolute safety we were seeking still eludes us. So we must establish priorities and accept risks.

In Holland we do not have to reckon with earthquakes and tornadoes,our dwellings are not built to burglar-proof and bomb-proof standards. But there are numerous regulations concerning the causes and consequences of gas explosions and fire.

One man may take refuge in a multitude of insurance policies against every conceivable hazard, whereas another may have no insurance at all because he regards everything that may happen to him as an Act of God.

In what I have said above I have tried to highlight the relative and the subjective character of the relationship between safety and economy.

All the same, we must build safely but not in the sense of absolute safety. This approach requires the responsible engineer to have a high ethical conception of his profession and a high degree of expertise.

And since we, as a community, cannot have unlimited raw materials and energy at our disposal, we must also build economically.

2. Design and construction

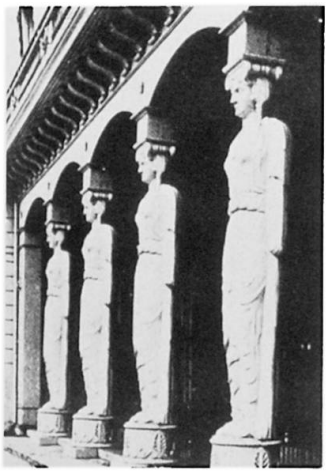
In their introductory report Fox and Timby have already called attention to the many phases involved, starting with the first decision to build, up to and including final demolition of the structure, in which considerably more influence can be exercised upon economy than is possible in the design and construction phase. Here, however, we must perforce confine ourselves to this last-mentioned phase.

Abeles, Bobrowsky and Bardhan-Roy in their paper give a clearly presented review of a number of important aspects of the design process. Their description of creative design shows how important already the first phase of the design activities is with regard to safety and economy. At that point hardly any design calculations have as yet been done; only the experience, expertise, insight and creativity of the designer play a part; thus it is the choice of the person, the responsible designer, that is the important thing here. In a later stage this preliminary design will be dimensioned and checked against existing rules and regulations.

In this context Abeles writes: "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."

In the design stage, economy does not yet mean the use of a minimum quantity of material or the lowest cost of construction. The designer is then still engaged in a process of weighing a great many aspects against one another, basing himself not only on the program of requirements, but also on relationships with the environment and taking account of the wishes, whether explicitly expressed or not, of the community.

When it comes to dimensioning and checking the design for its compliance with regulations, we encounter safety factors in the latter. These factors determine a lower limit of material consumption in the structure based on a predetermined design. Abeles rightly mentions emphatically the consequence of failure as one of the criteria that must affect the design. This aspect is not embodied in our regulations. The roof over a storage shed is judged in accordance with the same factors as the roof over a congress hall.



Ligtenberg has derived a simple approximate rule whereby the consequence of failure can be incorporated in the safety factors. This rule is capable of refinement, but to a designer that is not so important. What is important is that it provides him with a yardstick for every member of the loadbearing structure, namely, the ratio between the loss that occurs as a result of failure of a member (due to whatever cause) and the cost of that member. Let n denote this ratio. It will be most economic to design that member in such way that its probability of failure is $p_f = \frac{1}{10n}$.

If no risk to human life is involved, it is possible to make a fairly objective assessment of loss due to structural failure. But if there is such a risk, then an objective assessment of loss becomes impossible and much will depend on the structural engineer's sense of ethical responsibility. If we have sufficient statistical data of the loading on the member in question and of influences such as fire, explosion, etc., and if we know the resistance of this member to all those influences, we can establish a relationship between the probability of failure p_f and the load factor applicable to the member. We shall thus have returned to a deterministic analysis, but now a very realistic and economical one.

Subject to establishing a number of boundary conditions we can thus, for frequently encountered structural members in particular types of structure, lay down safety factors which ensure optimum dimensions of the members on the basis of a given design.

There are also other requirements, however. For example, if the safety factors for concrete floors are reduced, the designer will find himself in conflict with requirements as to permissible deflection and cracking.

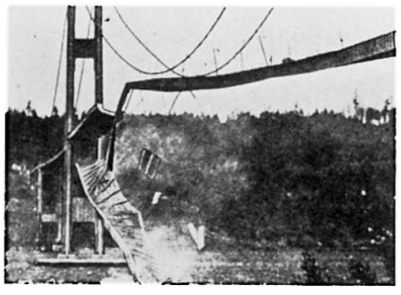
It has been calculated that in the Netherlands the lowering of the safety factor for concrete structures from 1.8 to 1.7 has achieved an annual saving of about 0.5% on the amount spent per year on all concrete structures. Within the overall process and for the community as a whole this is a fraction of what could be saved in total without detriment to safety. A considerable amount of research is being done into the methods of checking or monitoring structures with regard to safety. Let us keep these things simple in cases where frequently encountered normal structures are concerned, for which a good deal of experience is available.

Simple regulations and simple structural forms can effect quite significant savings on the cost of design, construction and materials. In most instances these normal run-of-the-mill structures are not the ones that the highly qualified structural engineers are called upon to design and detail.

Heynisch shows how regulations may sometimes be too simple, however, by rather indiscriminately lumping everything together. Such regulations do not enable economies to be effected; they are illogical. Regulations should give clear insight into the how and the why; they can quite legitimately stimulate the user to think for himself. Designing is more than just allowing oneself to be led tamely along by regulations.

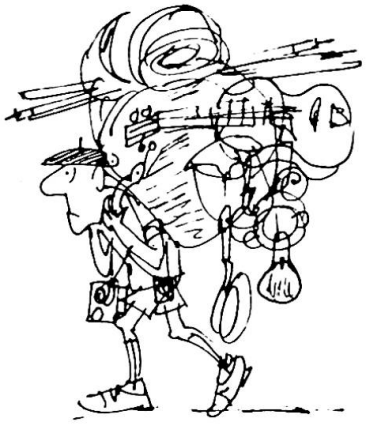
Although industrialized building may - also in terms of safety and economy - be better amenable to probabilistic design considerations and experiments, it is also vulnerable to the dangers threatening every "monoculture". There is a higher degree of susceptibility to errors. In Holland a very considerable amount of loss was sustained when a factory producing floor slabs supplied, over a long period of time, slabs with an excessive voids content and too high a content of calcium chloride in the concrete. These defects have lately begun to manifest themselves in places, requiring remedial action. In theory this corrosion may continue in the years ahead. All the slabs have been installed in buildings.

The paper by Kiyoshi Muto and Masayuki Nagata also presents a good example of not adhering to in themselves simple rules if a particular design entails a considerable increase of scale in relation to that with which the designer is familiar. The dynamic effects of wind on suspension bridges in past decades did indeed teach us a lesson in this respect. The building on which these authors are carrying out their interesting research is not one that belongs to the normal built-up environment, but the process for arriving at decisions is a good example of a methodical procedure for checking a structure with regard to safety in resisting a particular kind of loading.



Dotreppe and Frangopol translate into graphs the in itself important distinction between what they call the "type chaine", comparable to series connection in electricity where the failure of one lamp causes all the others to go out, and the "type ductile", comparable to parallel connection. In the first-mentioned type, progressive collapse may occur, the safety of the individual

member being greater than that of the structure as a whole, whereas in the second type the safety of the whole is greater than that of its individual parts. This is a very important distinction, which every designer ought to be aware of and to take into account in his design.



Klingmüller indicates in principle a method of determining the probability of collapse of a structure. This probability, multiplied by the loss incurred, yields an amount which should be added to the price in order to arrive at the actual cost. As yet our knowledge is still too incomplete to make this into a generally practicable method. It directs our thinking to the probabilistic character of our safety considerations, and we may regard it as a pointer to penetrating further into this subject matter.

Mrazik studies structures in the limit state, basing himself on the rule that the strength minus the loading should exceed zero. This rule can be elaborated statistically. It thus provides better information than the approach based on the conventional permissible stresses or safety factors. His paper also presents a good example of the application of the new way of thinking.

3. Summary and conclusions

Out of respect for human life and well-being we must build safe structures. Yet our calculations must allow for risk. If we could so design buildings that collapse is preceded by a warning in good time, so that people have sufficient time to escape, safety can be studied as an economic problem. The decisions which could effect the greatest savings to the community are usually made in the preliminary design phase by people who are not engineers.

In comparison with these savings the fairly small amounts that can be saved on the cost of a structure as a result of refinements in the design calculations often appear unrealistic. They are really of value only in so far as they give us better insight into structural safety and thus favourably affect our design.

The fairly recent publication by Matousek and Schneider of the E.T.H. (Federal Technological University), Zürich, entitled "Untersuchungen zur Struktur des Sicherheitsproblems bei Bauwerken" offers an interesting review of research into causes and consequences of damage affecting 800 buildings and other structures. The causes do not lie in load factors, overloading or statistically accountable deficiences in material properties. In all cases they are attributable to human action, inexpertness, taking too big risks in execution of the work, carelessness, etc. If we propose to talk about safety and economy, then we must turn our attention to those matters too.

Summary

Within the overall process and for the community as a whole the savings which can be reached in the calculation phase of the structure are a fraction of what could be saved in total without detriment to safety. Let us keep the things simple in cases where frequently encountered normal structures are concerned, for which a good deal of experience is available.

The causes of damage are in most cases attributable to human action, inexpertness, carelessness, etc.

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Comments by the Author of the Introductory Report

Remarques de l'auteur du rapport introductif

Bemerkungen des Verfassers des Einführungsberichtes

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Serviceability and Maintenance

The Introductory Report for Theme 1c, Serviceability and Maintenance, presents an overview of the various life-cycle considerations applicable to structures from their conception as a mere idea to their eventual demolition or replacement. Also discussed is how planners, designers, materials suppliers, equipment specialists and builders can best serve the interests of the owner and public.

The five primary life-cycle aspects of concern are functional service, capacity to serve, environmental effects, time schedules and cost-benefit relations. Each of these primary aspects is comprised of many elements which must be weighed and evaluated taking into account human, social, economic and monetary values as well as the technical considerations.

It is concluded that the optimization of the five primary life-cycle aspects can be achieved at relatively little cost and that favorable serviceability and maintenance depend on adequate planning, design and construction.

One submission was selected under Theme 1c, Serviceability and Maintenance, for publication in the Preliminary Report. In the accepted paper Mr. Kuesel clearly describes the four phases that evolved to replace an old 2-lane bridge crossing the 7.4 km wide James River in Virginia, U.S.A. The bridge was not only obsolete but more importantly was in an advanced state of deterioration and in danger of collapsing.

During phase one the existing bridge was repaired sufficiently to provide a safe structure until a new parallel 2-lane trestle structure could be completed under phase two. By means of temporary crossover structures traffic utilized the new trestle span and the existing 800 m channel spans. The completion of phase two eliminated the need for use of 90% of the existing deteriorated bridge.

Under phase three a new four-lane channel crossing was constructed. The bridge was completed during phase four with the addition of a second two-lane trestle.

A noteworthy feature of the new trestle span was the precasting of the deck section in a monolithic unit 23 m long, consisting of four longitudinal beams, transverse diaphragms and a 20 cm deck slab. Heavy marine equipment placed the units which weighed 230 tons each. Extra prestressing steel was utilized to obtain a uniform stress across the section under dead load plus prestress thus eliminating creep flexural deflections and ensuring a smooth riding surface.

The replacement schedule was very carefully planned by the Consultants to allow for the completion of phases one and two within the limited financial resources available. Phases three and four could then be deferred until the necessary funds became available to complete them.

Consideration of life-cycle aspects of a project usually end with the demolition of a structure. However, new bridges usually become important and vital links in a highway or railroad transportation system and as such when the time comes for their replacement, in part or total, due to obsolescence, consideration must be given to maintaining a continuous traffic flow during the construction period. This constraint of maintaining service should become an important element of any life-cycle analysis.

Selection of Hollow Steel Plate Deck for Floor System of Long-Span Suspension Bridges

Choix d'une section métallique fermée pour le tablier des ponts suspendus de longue portée

Zur Wahl eines stählernen Hohlkastens für das Deckensystem weitgespannter Hängebrücken

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1. Introduction

This contribution on planning of structures and its relationship with construction methods is for the additional study on the "Planning of Floor System at Long-Span Suspension Bridges" which was classified in Theme IIb in the Preliminary Report.

When a floor system is planned at a long-span suspension bridge provided with stiffening truss girders, many kinds of floor system can be proposed as discussed in the Preliminary Report.

In this paper the relationship of planning of the floor system of hollow steel plate deck with its fabrication method examined in detail.

2. Evaluation of Floor System

In the Preliminary Report, the authors examined several possible floor systems, which are a floor system, (1) with reinforced concrete slab, (2) with closed steel grating floor, (3) with precast concrete steel grating floor, (4) with prefabricated steel deck plate sandwiching concrete, (5) with prefabricated composite girder, (6) with orthotropic steel plate deck, and (7) with hollow steel plate deck, to evaluate which floor system would be the most suitable for a long-span suspension bridge with stiffening trusses. At the study, structural features of various floor systems were examined and compared with one another on such conditions as fabrication, erection, maintenance, economy and so on, by making a decision matrix.

As a result, it might be considered to be generally advantageous to provide the bridge with a lighter floor system instead of a heavier concrete floor system. Furthermore, among the steel floor systems a new type of hollow steel plate deck was proved to be better than an ordinary orthotropic steel plate deck.

Prof. F. Moses pointed out at his General Report at the Congress that the present study was interesting from the viewpoint that various types of deck for the floor system were examined about their cost and weight with comparative designs, and also

about their performance or function with a decision matrix, at the decision process.

However, it is quite necessary to examine the hollow steel plate deck in more detail to know if there might be some practical problems at its fabrication and erection.

3. Fabrication of Hollow Steel Plate Deck

Hollow steel plate deck developed by the authors has such a cross section as shown in Photo. 1, and consists of two face plates and core plates which are installed diagonally between face plates.

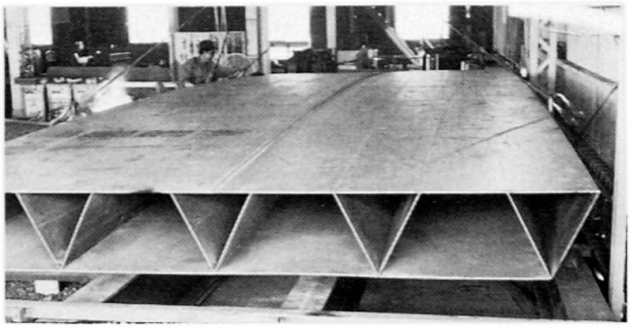
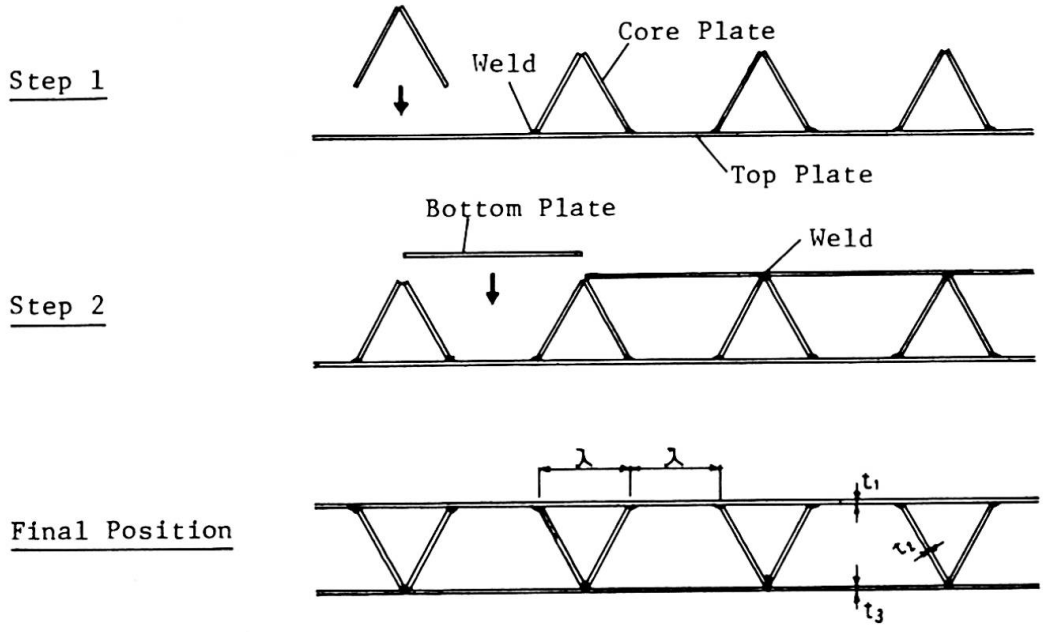


Photo. 1 Section of Hollow Steel Plate Deck

Fig. 1 is illustrating the fabrication sequence of this deck. The fabrication operation is done by reversing a top plate and a bottom plate. First of all, core plates are welded in a triangular form to a steel plate which will serve as a top face plate. Then, band-type bottom face plates are welded to the top of triangularly-formed core plates.



Note: For Highway Bridge,
Steel Plate Thickness:

Top Plate	t_1 =	12 mm
Core Plate	t_2 =	8 mm
Bottom Plate	t_3 =	9 mm
Interval of Core Plate	$\lambda \leq$	340 mm

Fig. 1 Fabrication Sequence of Hollow Steel Plate Deck

Photo. 2 shows welding of core plates to a top face plate by an automatic welding machine.

Photo. 3 gives a picture of a hollow steel plate deck for a foot-way bridge just before the fabrication is over at a shop. In this case, it has a span length of 20m, width of 3m, depth of 40cm and it was designed for a uniform live-load of 500kg/m².

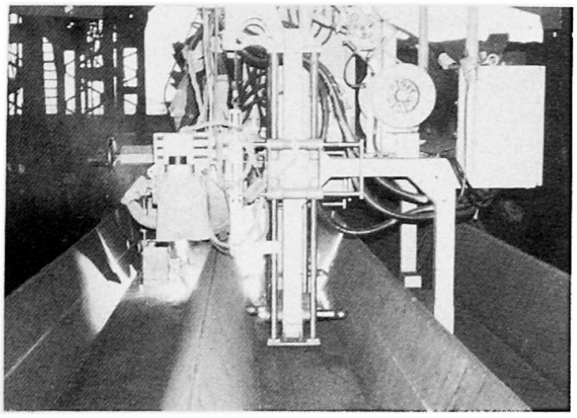


Photo. 2 Welding Operation

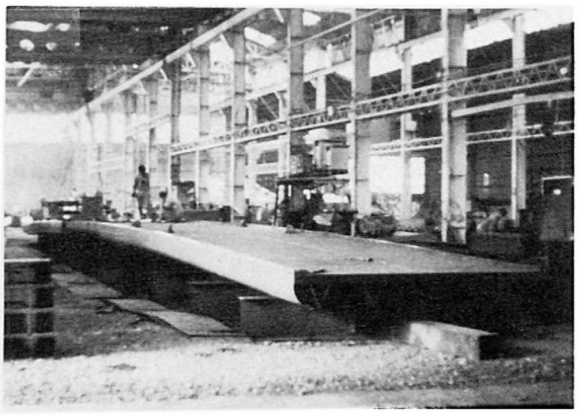


Photo. 3 Overall View of Hollow Steel Plate Deck

4. Features and Problems of Hollow Steel Plate Deck

Since the hollow steel plate deck provides a floor with great lateral and torsional rigidities, it has a greater load-carrying capacity as verified by loading tests. Also, Mr. G. F. Fox pointed out at his Summary Report at the Congress that this was an interesting structure because of its greater shearing resistance.

In the case of planning for a roadway bridge, its height can be reduced from one fortieth to one fiftieth of its span length. To apply this deck to a floor system at a suspension bridge with stiffening trusses, it can be set on main cross beams of the trusses directly without stringers. Therefore, it might be advantageous for a floor system of long-span suspension bridge from the point of aerodynamic stability of a bridge, because of a small height of this floor system compared with an ordinary floor system using stringers. However, as Dr. W. C. Brown discussed at the Free Discussion for Theme IIb in the Congress, the aerodynamic stability of floor has to be examined thoroughly not only from their local behavier, but from overall bridge behavier.

As mentioned above, since at the fabrication of this deck, welding operation for major components of the deck is performed only in one direction, less welding distorsion and easier fabrication can be observed. Since it makes a closed section, however, it is difficult to make inspection for welded parts after fabricated.

There will be still uncertainties in the construction method of this floor system at a long-span suspension bridge. For example, there will be some problems at the erection of the deck and at its connection with supporting shoes on cross beams and with expansion joints, etc.. Therefore, to make a more reliable decision a more detailed examination has to be done through more detailed practical informations together with application of a probabilistic approach to uncertainties.

5. Conclusion

At the present study on the selection of a hollow steel plate deck for a floor system at a long-span suspension bridge with stiffening trusses, greater load-carrying capacity, better aerodynamic stability due to its small height, easier prefabrication and economy compared with ordinary floor systems, could be confirmed in a certain way, supporting the engineering judgement at the proposed decision processes in the Preliminary Report.

However, there will be still uncertainties at the construction method of this floor system at a long-span suspension bridge. More detailed evaluation has to be done through more detailed engineering informations for various uncertainties to make a more reliable decision.

SUMMARY

The authors have shown - in the Preliminary Report - the advantages of a hollow steel plate deck instead of the ordinary floor systems, for a long-span suspension bridge with stiffening trusses. The present study is intended for evaluating; the hollow steel plate deck for its fabrication, features and problems.

RESUME

Les auteurs ont montré - dans le Rapport Préliminaire - les avantages d'un tablier métallique de section fermée au lieu des tabliers ordinaires, dans le cas de ponts suspendus de longue portée, en poutres à treillis. On présente ici les caractéristiques et les problèmes liés à la fabrication du tablier métallique de section fermée.

ZUSAMMENFASSUNG

Im Vorbericht haben die Autoren die Vorteile eines als Pfahlhohlkasten ausgebildeten Fahrbahnträgers gegenüber anderen Deckensystemen dargelegt für den Fall weitgespannter Hängebrücken. Diese Studie berichtet über die charakteristischen Eigenschaften und die Herstellungsprobleme des obgenannten Deckensystems.

Zwei Baukonstruktionen im Internationalen Konferenzgebäude der UN-City in Wien

Two Types of Construction in the International Conference Building in the UN-City in Vienna

Deux types de construction dans le Centre International des Conférences de la Cité des Nations Unies, à Vienne

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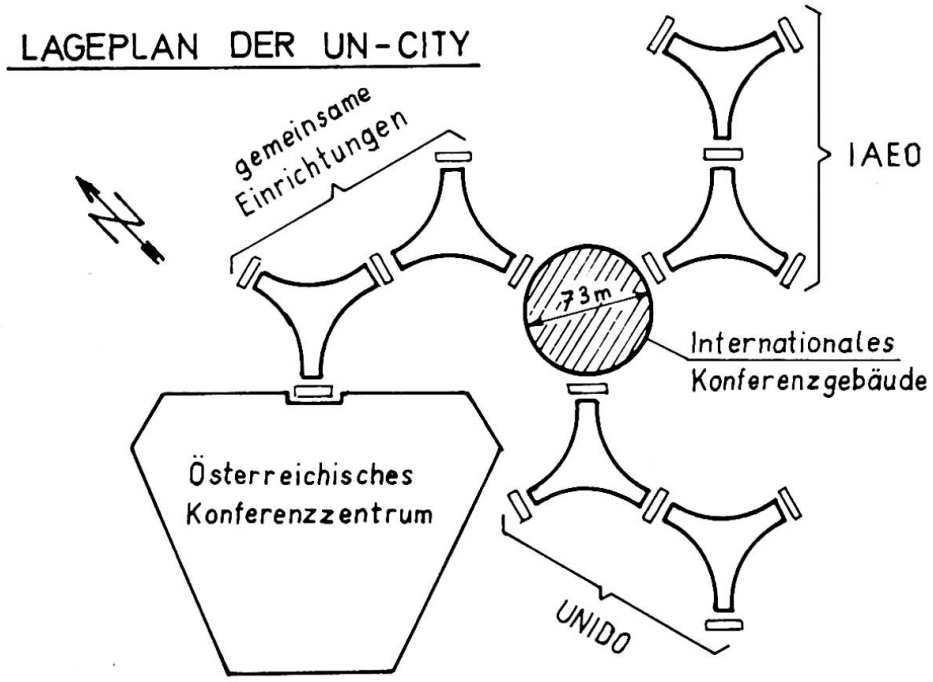
Dipl. Ing.
Vöest-Alpine A.G.
Wien, Oesterreich

1. Allgemeines

Die von Dr. AHORNER im Vorbericht beschriebenen Amtssitzgebäude der UN-City in Wien werden als Bürohäuser für zwei UN-Organisationen gebaut. Sie sind zur Gänze in Stahlbeton konzipiert. Zusätzlich zu diesen Bürogebäuden wird im Rahmen dieses Großbauvorhabens auch ein für beide Organisationen dienendes "Internationales Konferenzgebäude" errichtet, bei dem eine interessante Stahlkonstruktion verwendet wurde.

Dieses Gebäude liegt in der Mitte der Y-förmigen Bürohochhäuser und hat einen kreisförmigen Grundriß mit einem Durchmesser von 73 m (siehe unten dargestellten Lageplan sowie auch die Modellaufnahme auf Seite 3 des Vorberichtes).

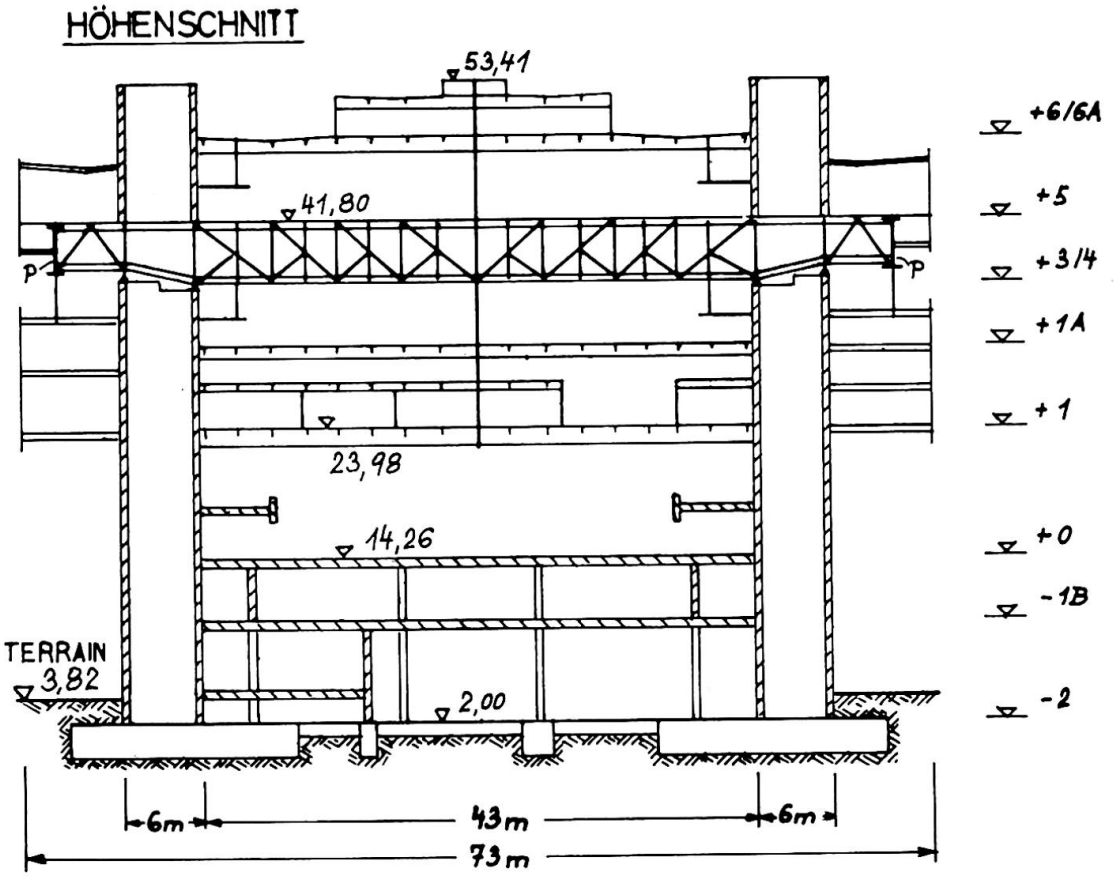


UNIDO (Kommission für die Industrialisierung in Entwicklungsländern)

IAEO (Internationale Atomenergie Kommission)

2. Konstruktiver Aufbau

Da das Bauwerk von beiden Organisationsbereichen niveaugleich benützt werden soll, wurden die verschieden großen Konferenzräume in die oberen Geschoße verlegt (siehe Höhenschnitt). Sie befinden sich innerhalb eines Kreisringes, in dem die Stiegenhäuser und die Schächte für Klimaleitungen und Aufzüge untergebracht sind (siehe Grundriß).



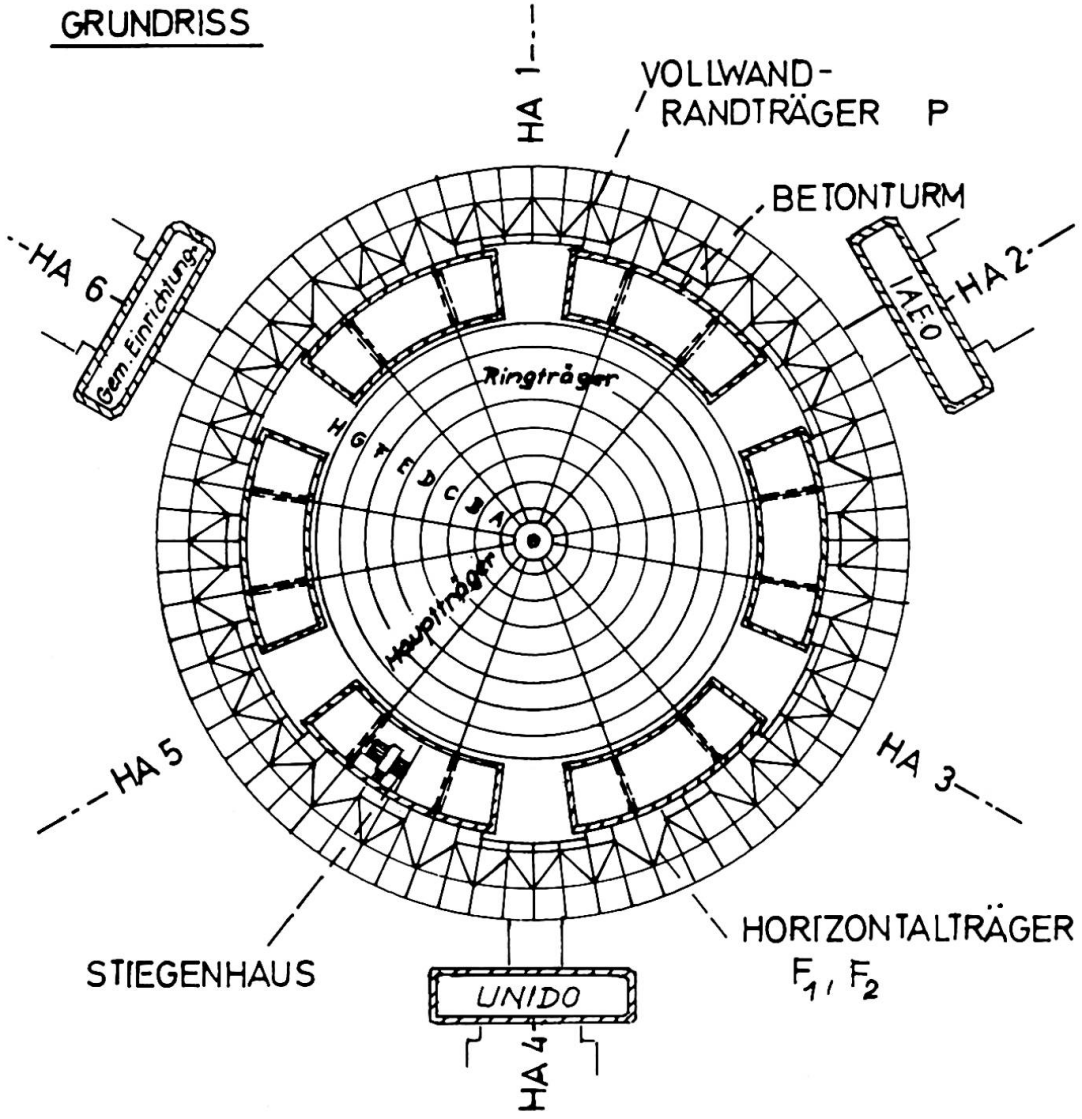
Der gesamte Außenbereich, in dem die zu den Konferenzräumen zugehörigen Nebenräume, Foyer's und Terrassen angeordnet sind, befindet sich daher nur im oberen Bereich und ist demnach auskragend. Die insgesamt sechs Betontürme im Bereich des Kreisringes übertragen alle Vertikal- und Horizontallasten nach unten. Sie sind voneinander unabhängig und haben getrennte Flachgründungen.

Zwischen den oberen und den unteren Etagen im Innenbereich liegt ein stützenloser Raum, der die gesamte innere Kreisfläche innerhalb der Betontürme ausfüllt. Aus diesem Grunde müssen alle oberen Geschoße in ihrer Gesamtheit über einen inneren Durchmesser von c. 43 m frei überbrückt werden. Auf Wunsch des Architekten durften diese oberen Etagen nur im Kreismittelpunkt durch ein vertikales Element miteinander verbunden werden. Durch die niveaugleichen Übergänge vom Konferenzgebäude zu den anschließenden Bürogebäuden ist nicht nur die Lage sondern auch die Konstruktionshöhe der Decken vorgegeben. Dadurch bedingt, entstand für die Bauhöhe der Deckenträger eine für die gewünschten Spannweiten sehr niedrige Bauhöhe.

Nur durch die Anordnung einer Stahlkonstruktion im oberen Geschoßbereich konnten die Vorstellungen des Architekten möglichst wirtschaftlich verwirklicht werden.

3. Beschreibung der Stahlkonstruktion

Für die Anordnung der Haupttragkonstruktion steht nur das Technische Geschoß, in dem die Klimazentrale untergebracht wird, zwischen den Ebenen + 3/4 und + 5 mit etwa 5 m Höhe zur Verfügung. Diese Geschoßhöhe ermöglicht die Verwendung von Fachwerkträgern mit genügend großer Bauhöhe. Als Tragkonstruktion wurden sechs sich kreuzende, radial verlegte Fachwerkträger mit Hilfsausfachung gewählt, wodurch sich die Austeilung der Hauptträger in Winkeln von abwechselnd 20° beziehungsweise 40° ergab.



Um die Belastungen aus dem Außenbereich aufnehmen zu können, werden die Hauptträger durch entsprechende Aussparungen der Betontürme hindurchgeführt. An ihren auskragenden Enden werden alle stählernen Geschoßdecken außerhalb der Betontürme unterstützt oder aufgehängt. Die Stahldecken des inneren Bereiches werden im Kreuzungspunkt der Fachwerkträger - also im Kreismittelpunkt - unterstützt.

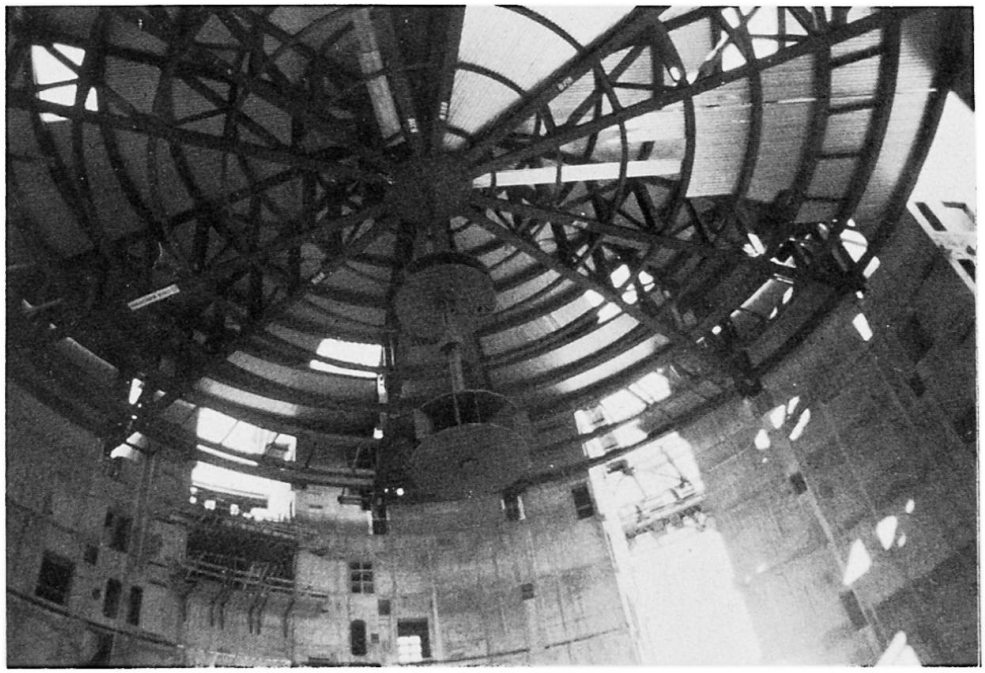
Um für die waagrecht wirkenden Kräfte eine möglichst gleichmäßige Übertragung auf die Türme zu erreichen, wurden unter den 60 cm hohen Kipplagern, 10 cm dicke Neopreneschichten verlegt. Zur Aufnahme der Auflagerdrücke konnten je Trägerhälfte zwei Lager

- ein äußeres und ein inneres - in einem Abstand von 6 m angeordnet werden. Der Aufstellungsort war durch die Lage der Stiegenhausmauer vorgegeben. Um für beide Lager eine möglichst gleichmäßige Belastung zu erreichen, wurden in den Fachwerkträgern innerhalb der beiden Lager Ausfachungen nicht vorgesehen. Dadurch konnte das Tragwerk im inneren Bereich wesentlich entlastet werden.

Für den Entwurf von wirtschaftlichem Vorteil war, daß die ortsansäßigen Stahlbaufirmen in der Lage waren, die Fachwerkträgerhälften mit einer Bauhöhe von 5,4 m in Längen von 22 m fertiggestellt vom Werk an die Baustelle zu transportieren.

Die zentrale Hängestange zur Unterstützung der unter dem Traggeschoß befindlichen Decken leitet ca. 2000 Mp in den Knotenpunkt der Fachwerkträgeruntergurte ein. Sie ist dort eingeschweißt, ist geschmiedet und hat einen kreisrunden Querschnitt mit nur 36 cm Durchmesser.

Traggeschoß mit Hängesäule



Die Deckenkonstruktion im Bereich der Gurte der Fachwerkträger besteht aus kreisringförmigen Hohlträgern, die an den Knoten des Fachwerkes angeschlossen sind. Über diesen Ringträgern liegen Trapezbleche, die als verlorene Schalung für den Leichtbeton dienen und gleichzeitig das frühzeitige Begehen nach der Montage ermöglichen. In allen Ebenen ist die Deckenkonstruktion gleich ausgebildet, nur sind hier die Hauptträger als Vollwandträger angeordnet. Sie lagern einerseits beweglich auf den Stiegenhaustürmen auf (Neoprenelager) und sind andererseits mittels eines kreisförmigen Plateaus an die Hängestange beziehungsweise Zentralsäule in Raummitte so angeschlossen, daß eine Durchlaufwirkung für die Radialträger möglich ist.

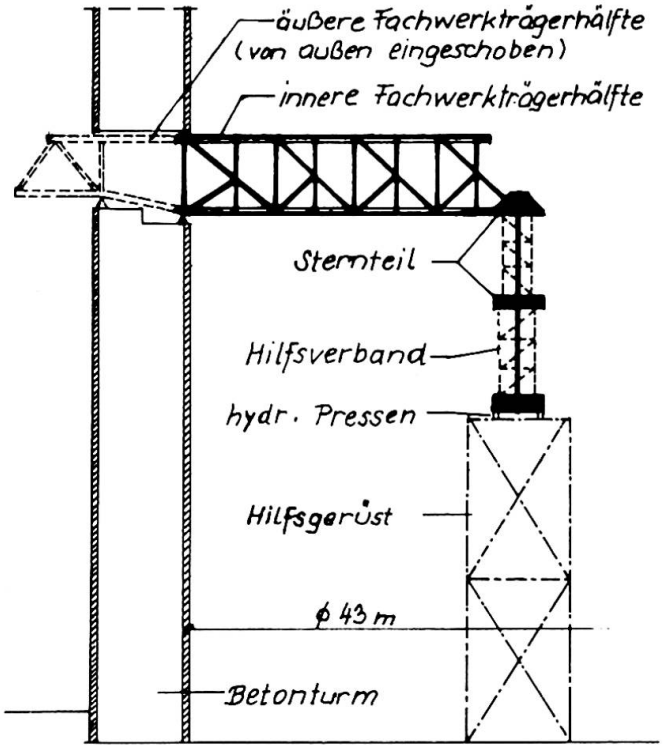
Am Ende der Hauptträger-Kragarme ist ein polygonal geknickter 4 m hoher Vollwandträger angeschlossen, der als Kreisringträger die anteiligen Lasten aller Decken außerhalb der Türme aufnimmt und an die Hauptträger abgibt.

Vollwandträger-
Kreisring außen

4. Beschreibung der Betonkonstruktion

Die drei untersten Etagen im inneren Bereich weisen zahlreiche Säulen auf, wodurch sich eine Ausführung in Stahlbeton als besonders wirtschaftlich zeigte. Stahlbeton wurde auch für die Stiegenhaustürme sowie für die zwischen den Türmen liegenden Decken gewählt. Die Galerie im Konferenzsaal auf Ebene 0 war aus architektonischen Gründen ebenfalls in Stahlbeton erwünscht. Deswegen mußten sogar einzelne aus den Stiegenhauswänden auskragende Spannbetonträger geplant werden, in denen die Spannglieder erst nach Beendigung der Stahlbaumontage - um diese nicht zu behindern - eingebaut werden konnten. Zwischen den Kragträgern spannen sich Ortbetonplatten.

MONTAGE DER HAUPTTRÄGERHÄLFTEN



5. Reihenfolge beim Einbau der verschiedenen Baukonstruktionen

Um die beiden verschiedenen Bauweisen übereinander überhaupt ermöglichen zu können, mußte die Stahlbaumontage dem Einbau der unteren Stahlbetongeschoße vorgezogen werden.

Die schweren Bauteile der Stahlkonstruktion mußten im Innenbereich mit einem 300 t Autokran, der am gewachsenen Boden stand, eingehoben werden. Der Einbau von Betondecken hätte die Montage der Stahlkonstruktion nur behindert. Innerhalb der bereits bis zum Technikgeschoß errichteten Betontürme mußte mit der Montage der tragenden Fachwerkträger begonnen werden. Dies war wegen der zentrischen Symmetrie des Tragsystems nur mit Hilfe einer provisorischen

Unterstützung im Sternmittelpunkt möglich. Da aber gerade an dieser Stelle die Hängestange an den Fachwerkträgergurten hängt, entschied man sich dazu, Letztere auf ein Hilfsgerüst aufzustellen und durch Einbau von Hilfsverbänden für den Montagevorgang drucksteif zu machen. Durch Zwischenschaltung von hydraulischen Pressen konnte die Hängestange vor dem Ausbau des Hilfsgerüstes wieder entlastet werden.

Anschließend wurden die unter dem Traggeschoß befindlichen Geschoßdecken mit einem Teleskopkran von unten eingebaut. Erst nach Abschluß dieser Montagearbeiten wurde die Herstellung der untersten Etagen in Stahlbeton sowie die Fertigstellung aller restlichen Stahlbereiche außerhalb der Stiegenhaustürme und über dem Traggeschoß vorgenommen.

Der gesamte Außenbereich konnte unabhängig davon mit Autokranen montiert werden. Durch Verwendung von Neoprenelagern können die gegenseitigen Verschiebungen zwischen Beton- und Stahlkonstruktionsteilen beherrscht werden.

Die gesamten tragenden Stahlkonstruktionen wurden von den beiden größten Stahlbaufirmen in Österreich geliefert und montiert.

ZUSAMMENFASSUNG

Bei der Planung des Internationalen Konferenzgebäudes für die UN-City in Wien hat sich aus architektonischen Gründen die Notwendigkeit ergeben, zwei verschiedene Bauweisen in einem Gebäude übereinander anzuordnen. Mit Hilfe von besonderen Montagemassnahmen musste die über der Betonkonstruktion liegende Stahlkonstruktion vor deren Einbau montiert werden.

SUMMARY

When the International Conference Building of the Vienna UN-City was being planned, its architecture gave rise to the necessity to provide two different types of construction, one above the other, within one and the same structure. With the aid of particular construction devices, the steel construction situated on top of the concrete structure had to be erected before the latter one.

RESUME

Lors de la conception du Centre International des Conférences de la Cité des Nations Unies, à Vienne, des raisons architecturales ont entraîné la nécessité de deux types de construction dans un seul bâtiment. A l'aide de méthodes spéciales de montage, la construction en acier reposant sur celle en béton a dû être montée avant que celle - ci ne soit mise en place.

The Cable-Net Cooling Tower at Schmehausen

La tour de refroidissement avec un réseau de câbles construit à Schmehausen

Der Seilnetzkühlturm in Schmehausen

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Prof. Dr.-Ing. Dr.-Ing. E.h.Dr.Techn. h.c.
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Stuttgart, GFR

The cable-net cooling tower, as described in the preliminary report Ia, pages 33 - 38, is at present nearing its successful completion.

I wish to summarize in short its basic principles and show some slides of the construction on site.

If a cable net with triangular meshes and a surface with negative (anticlastic, hyperbolic) curvature is prestressed, it behaves with respect to outer loads, i.e. here mainly to wind, as an ideal membrane shell. It is not subject to any bending, even under high and local wind gusts, since it has no bending stiffness. It is also not subject to buckling, since it always acts in tension. Horizontal spokes wheels, consisting of rings and radial ropes, are provided as stiffeners (Fig. 1).

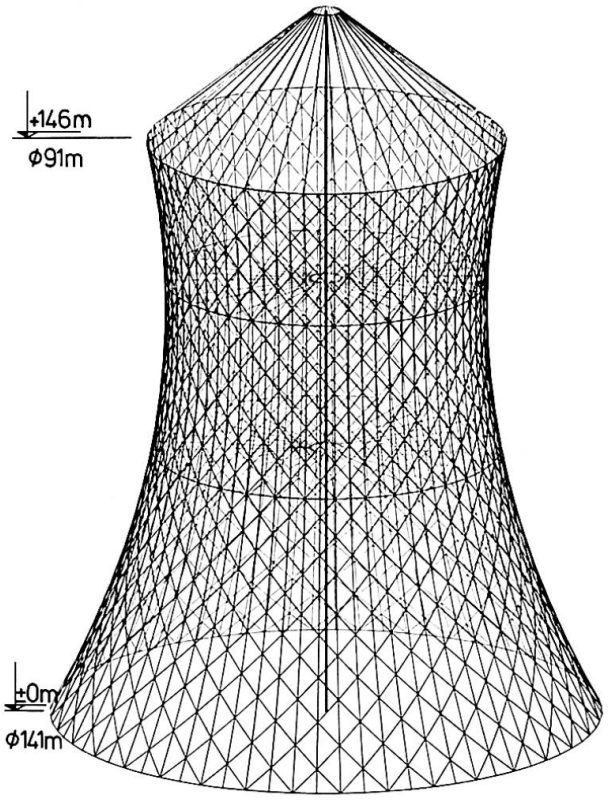

Fig. 1

Such a cable-net readily makes a cooling tower if it is covered with an air-tight cladding, leaving only its bottom part free up to a height as required for the air-inlet. The prestressing forces are introduced by tensioning the net vertically between a foundation ring at the bottom, anchored to the ground with prestressed soil anchors, and a steel ring at the top.

This ring at the top acts in compression, since it is suspended by inclined radial ropes from the top of a mast, which is placed at the center of the tower. Since this mast only is to maintain the prestress, it acts under pure compression and is preferably made from concrete. In Schmehausen the mast is 180 meters high, the diameter of the net being 91 meters on top and 141 meters at the base. The total net surface has an area of 46,000 m^2.

Construction at Schmehausen started with the tubular mast and the foundation ring simultaneously. The cable- and steel structures were designed such that they could be completely prefabricated in the work shops (according to an exact cutting pattern) and be lifted from ground into its final position. After the concrete works, the box steel ring of the future upper spokes wheel was assembled on ground as well as an inner ring surrounding the mast. After the radial ropes were fitted between these two rings, the whole assembly could be lifted (Fig. 2). For this purpose a lifting device was placed on top of the mast and connected by vertical steel rods with the inner steel ring.

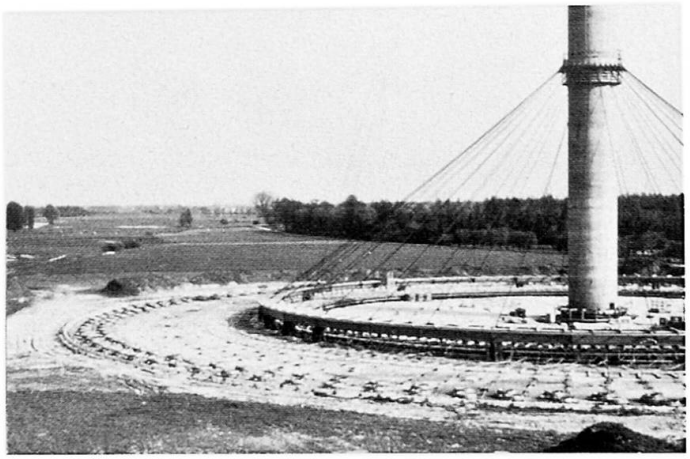

Fig. 2

After the outer ring just raised from ground, the cables of the net were fixed to it and unspooled whilst further lifting the ring. The net cables consist of two parallel strands each, with aluminum clamps press-fitted in the shop such that there is only one bolt required to connect the three layers of cables at each knot.

These bolts additionally serve for fixing aluminum ring beams, which are later to carry the cladding and which were also attached to the net on ground during the lifting process (Fig. 3).

Fig. 3

The continuous lifting process, which could cover 4 meters a day minimum, had to be interrupted shortly only twice, when the net-cables had to be fixed to the outer spokes-wheels, which were also assembled on ground and lifted with the net.

These rings carry trolleys, running around and like funiculars also up and down the cable net, to transport the corrugated aluminum sheets for the cladding into their final position (Fig. 4). (See page 38 of the preliminary report.)

The cladding could however only be fixed, after the net had reached its final height, was attached to the foundation ring and prestressed with the same (but strengthened) device as used for lifting - and finally after the lifting ring was permanently connected with the top of the mast (Fig. 5).

Fig. 4

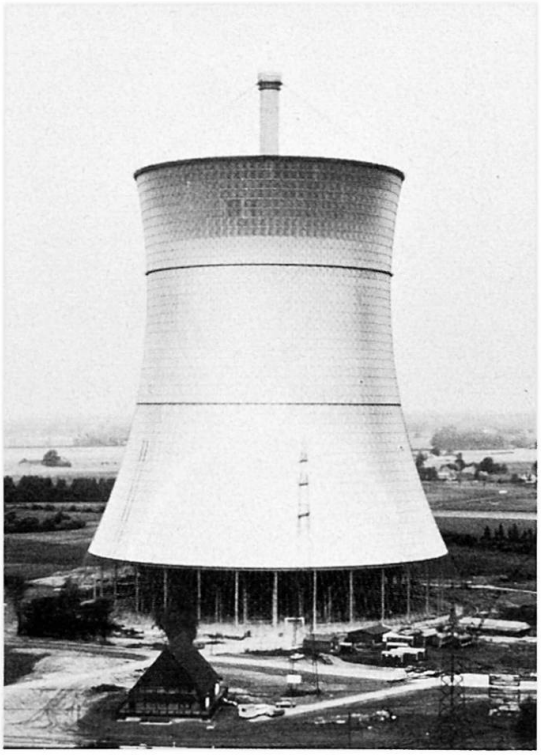

Fig. 5

The new type of a cooling tower with a triangular cable net was perceived by Jörg Schlaich and Günter Mayr, who are in charge of this structure in the consulting firm Leonhardt und Andrä, Stuttgart. Balcke-Dürr/GEA, Bochum, are the general contractors and Krupp Industrie- und Stahlbau, Goddelau, the contractors for the cable-net tower. With this system, cooling towers with even much larger sizes can be built.

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 Naturzugkühlturm 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 auf setzungsempfindlichen Böden.

Dynamic Design Criteria for Tall Buildings in Japan

Critères dynamiques pour le calcul de bâtiments de grande hauteur au Japon

Dynamische Entwurfskriterien für Hochhäuser in Japan

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1. Introduction

The earthquake resistant design is most important in the structural safety in Japan. The design of low buildings under 45 meters are performed by the conventional static method based on "Seismic Co-efficient" method, that is, percent G method.

On the other hand, a unique "Dynamic Design Method" has been established for the high-rise buildings over the height of 45 meters. The procedure of this new method is shown in Fig.1. In this method, the dynamic response analyses are conducted with the aid of a computer using observed records of strong earthquakes as input waves. The strength and deformation of the structure are checked covering the elastic and elastic-plastic ranges, and if the results on stress and strain do not satisfy the design criteria established by the designers to control response stresses and/or deformations, the structural design must be revised and checked again. This procedure is the so-called "feed-back" system.

This paper presents the dynamic design criteria in this procedure and the earthquake response values of several examples.

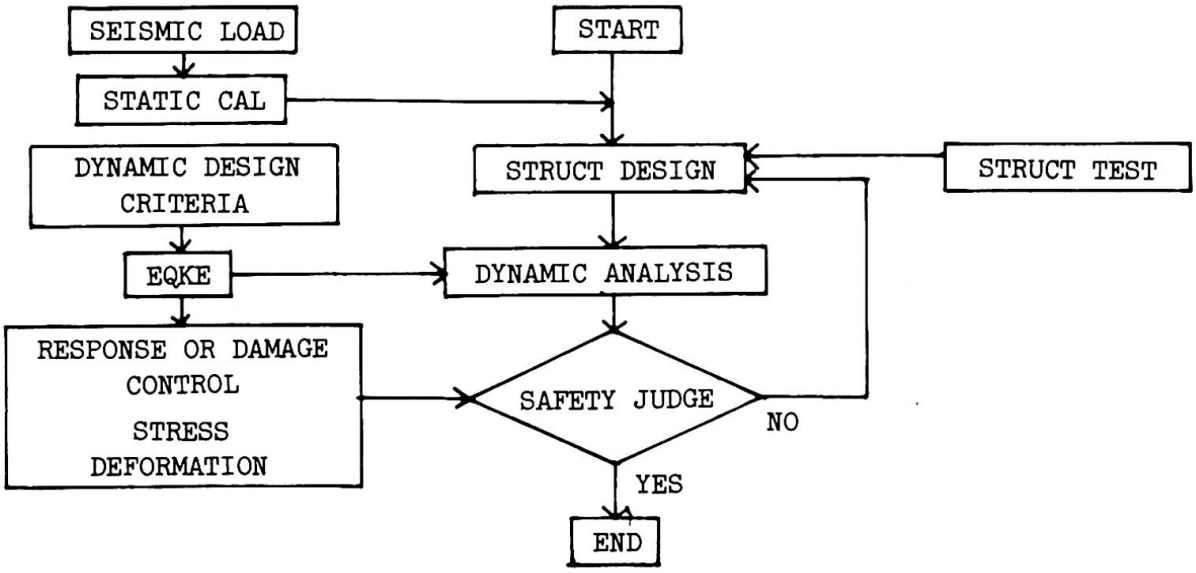


Fig.1 Dynamic Design Procedure of High-rise Building

2. Dynamic Design Criteria

Fig.2 shows the earthquake resistant design criteria set up by the authors' institute. Three degrees of earthquake intensity are assumed and against each degree, response limitations are respectively established.

For a minor or moderate earthquake, categorized as Class I, which occurs after and during which the behaviors of the structure can be confirmed by vibrations tests, the vibration must be controlled to be as small as possible to give no disturbance or discomfort to human. In other words, a high stiffness should be given to the structure in order to minimize the deformation. When an expected severe earthquakes hit, the stresses on all members of the structure must be less than the allowable values and also the secondary members should maintain safe condition. This category level, Class II, may be considered to correspond to the conventional code design level.

In the worst earthquake, hypothetically assumed as Class III, the structural and secondary members may exceed the elastic limit and suffer some damage but must not be severely damaged or collapse.

For the Tokyo region the authors usually assume the maximum acceleration of 0.2 - 0.3G as the Class II earthquake and 0.4 - 0.5G as the Class III earthquake.

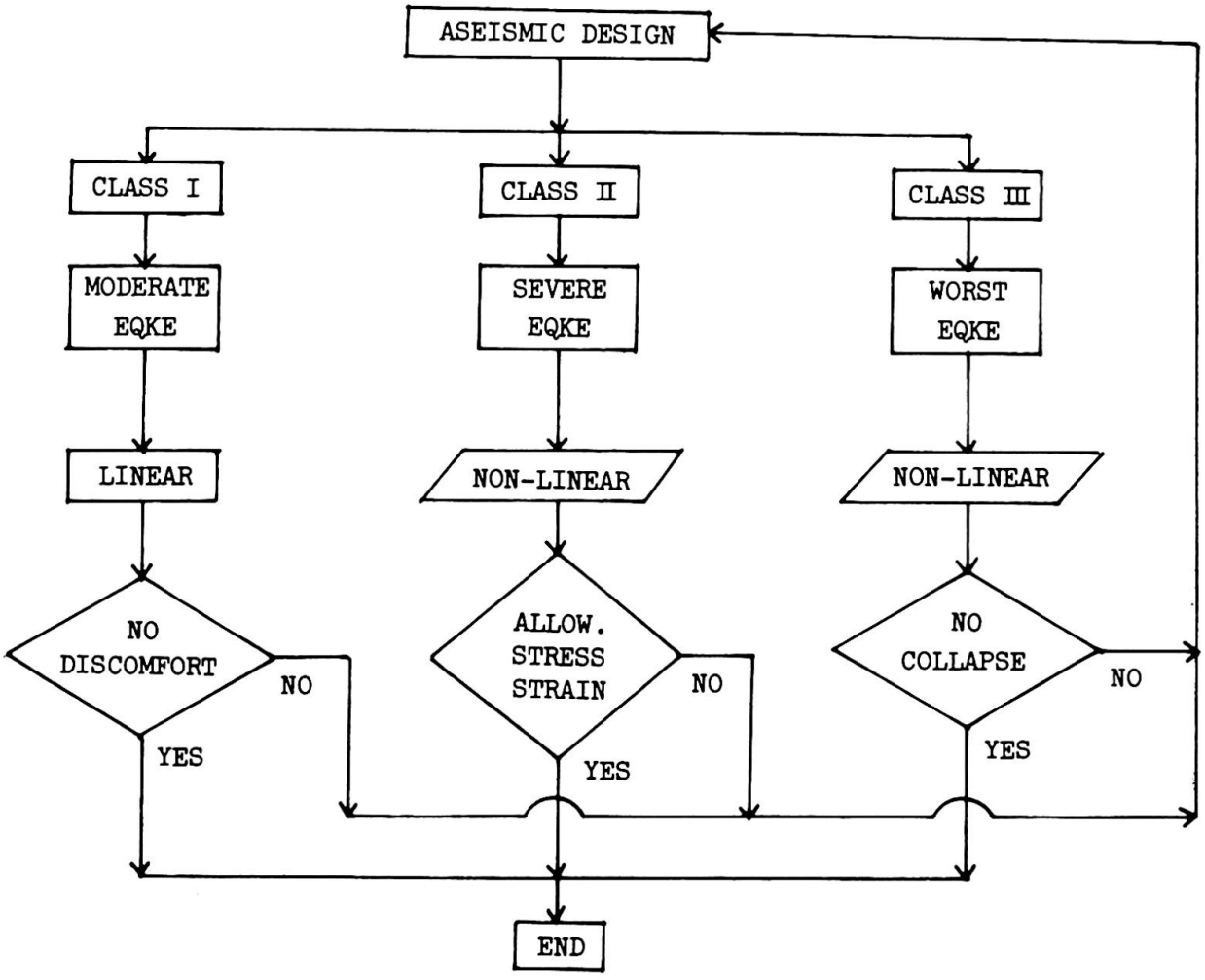


Fig.2 Dynamic Design Criteria Set Up by the Authors' Institute

3. Earthquake Response Values

The authors have undertaken to design and analyze many tall buildings in the way described above. In order to show the damage control, examples of maximum earthquake response values of three buildings are listed in Table 1.

The 60-story office building is under construction in Ikebukuro Subcenter of Tokyo and will be the tallest in Japan. (Photo 1) The earthquake resisting system of this building consists of rigidly connected steel framings with the special ductile reinforced concrete shear walls (slitted wall) as shown in Fig.3. Next 17-story office building consists of steel moment resisting framings only.

For these two high-rise buildings, the authors have assumed the maximum acceleration of 0.25G as the Class II earthquakes and 0.4G as the Class III ones in consideration of the important factor. All of these response values shown in Table 1 are appropriate for the design criteria.

Photo 2 shows the 4-story reinforced concrete garage building. Although the design of this building is in the category in which only the stress check is regulated by the conventional code, the dynamic analysis was performed especially in order to investigate the safety against earthquakes.

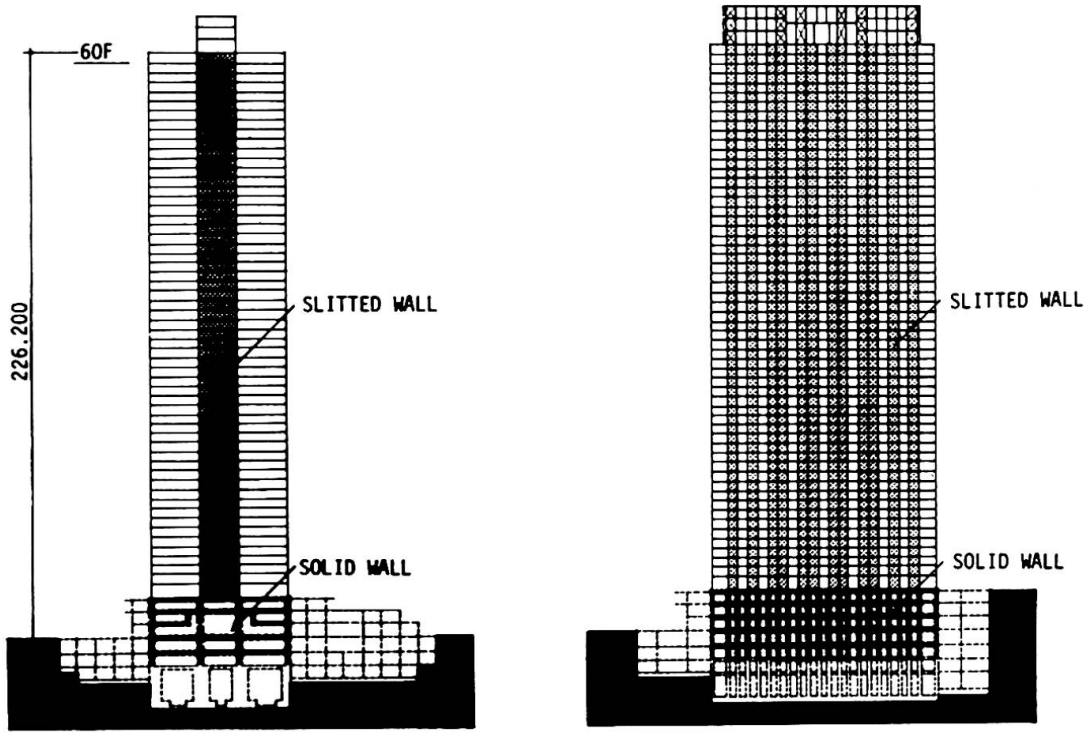
In this case the design criteria for tall building are not satisfied, but this building was designed to have sufficient ductility by placement of rebars in special arrangement shown in Fig.4.

Table 1 clarifies that for tall buildings the important problem in designing is the stiffness control, and consequently the tall buildings are given more than sufficient strength. On the other hand for low buildings it is important to control the deformability with satisfactory ductility as much as the strength.

Building (Structure)	Earthquake Intensity	Damage Control (Max. Value)		
		Strength	Story Defl. Angle	Ductility Factor
60-story Office (Steel framing & R.C. Slitted Shear Wall)	0.25G	Elastic	1/400	0.7
	0.40G	Partially Yield	1/260	1.3
17-story Office (Steel framing)	0.25G	Elastic	1/190	0.9
	0.40G	Partially Yield	1/110	1.4
4-story Garage (Moment Resisting R.C. framing)	0.20G	Yield	1/130	1.3
	0.40G	Yield	1/60	2.6

Table 1. Examples of Damage Control

Photo 1. Ikebukuro Subcenter Development (Right: 60-Story Office)



Transverse Direction Longitudinal Direction

Fig.3 Framing Sections of 60-Story Office Building

Photo 2. 4-Story Reinforced Concrete Garage Building

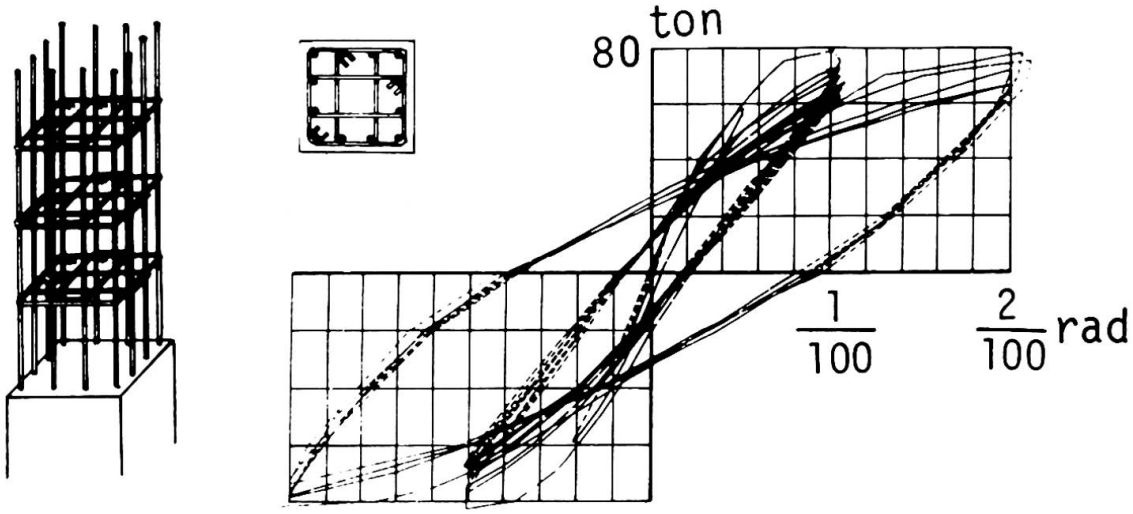


Fig.4 Shear-Deflection Hysteresis Loop of Tied Column Used in the 4-Story Garage Building

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. In this system dynamic design criteria are indispensable in order to evaluate the earthquake response values obtained from the dynamic analysis. Without those damage evaluations and controls, seismic design, in the true sense of the word, can never be established.

RESUME

La méthode dynamique par itérations successives est, au Japon, reconnue comme la plus avancée pour l'étude des bâtiments de plus de 45 mètres de hauteur. La connaissance des critères dynamiques pour le calcul est indispensable afin d'évaluer les réponses sismiques obtenues par l'analyse dynamique. Sans évaluation des dégâts et sans contrôle, une étude parasismique, aus sens propre du terme, ne pourra jamais être mise sur pied.

ZUSAMMENFASSUNG

Die iterative dynamische Berechnungsmethode wurde in Japan als die fortschrittlichste Methode bei Hochhäusern über 45 m angenommen. In dieser Methode ist die Kenntnis der dynamischen Kriterien für die Berechnung unerlässlich, um die Wirkung des Erdbebens nach der dynamischen Analyse abzuschätzen. Ohne diese Schadenermittlungen und Kontrollen ist eine seismische Berechnung und Bemessung undenkbar.

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Considérations sur la sécurité par rapport à différents types de comportements

Sicherheitsbetrachtungen mit Rücksicht auf verschiedene Verhaltenstypen

Considerations on Safety in Relation to Different Types of Behaviour

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1. INTRODUCTION

Dans notre contribution au Rapport Préliminaire [1], nous avons examiné, du point de vue probabiliste, deux types de comportements structuraux : le comportement du type "chaîne" et le comportement du type ductile. Les raisonnements ont été effectués à partir de modèles très schématiques, qui ne permettent de couvrir qu'une gamme assez restreinte de problèmes. C'est le cas, par exemple, de l'étude de la sécurité des structures isostatiques où la ruine d'un élément amène la ruine de l'ensemble de la structure (comportement schématisé par le modèle série - figure 1.-), et de l'étude de la sécurité des structures hyperstatiques dans lesquelles se produit une adaptation plastique entre toutes les sections critiques avant la ruine (comportement schématisé par le modèle parallèle ductile-figure 2.-).

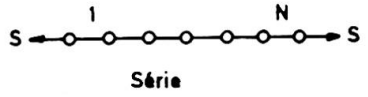

Figure 1

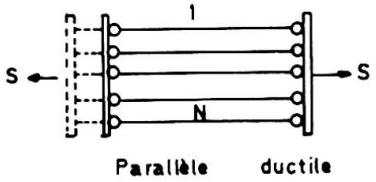

Figure 2

L'objet de cette discussion est de mettre en évidence l'existence d'autres modèles permettant d'étudier la sécurité de constructions dont le comportement est plus complexe, et de présenter un certain nombre de considérations sur ces modèles.

Parmi ceux-ci, on peut citer le modèle parallèle fragile (figure 3), et le modèle mixte (figure 4) constitué par l'association de capacités portantes, en partie en série et en partie en parallèle.

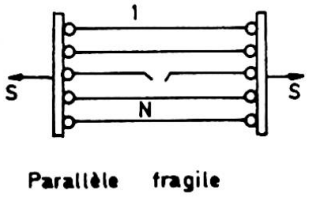

Figure 3

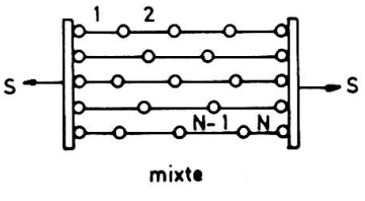

Figure 4

2. COMPORTEMENT DU TYPE PARALLELE FRAGILE.

Ce type de comportement est caractéristique des constructions hyperstatiques dont les sections ne possèdent pas de capacité de déformation plastique avant la ruine, comme par exemple les structures en béton précontraint. La rupture d'un élément n'engendre pas la ruine d'ensemble de la structure, mais amène une majoration des efforts dans les éléments restants et conduit, par conséquent, à une diminution de la probabilité de survie de ces éléments.

Une des difficultés de l'évaluation de la probabilité globale de survie de la structure réside dans le fait qu'on doit utiliser la théorie des probabilités conditionnelles et considérer l'ensemble des chemins conduisant à la ruine de la structure ("failure paths").

La survie du modèle exige la survie à tous les N! chemins de ruine possibles. La probabilité globale de survie du modèle parallèle fragile (voir figure 3) s'écrit donc [4], [8] :

$$P_{(+)} = P\left(\bigcap_{i=1}^{N!} E_{(+)i}\right) \qquad (1)$$

où $E_{(+)i}$ représente l'événement survie par rapport au i-ème chemin de rupture

$N!$ représente le nombre total de chemins de ruine possibles

La probabilité d'apparition du i-ème chemin de ruine, qui pourrait, par exemple, être constitué par la rupture des éléments dans l'ordre 1, 2, 3,..,N, est donnée par l'expression suivante :

$$P_{(-)i} = P_{(-)i}^{(1)} \cdot P_{(-)i}^{(2/1)} \cdot \ldots \cdot P_{(-)i}^{(k/1,2,..,k-1)} \cdot \ldots \cdot P_{(-)i}^{(N/1,2,..,N-1)} \qquad (2)$$

où $P_{(-)i}^{(k/1,2,..,k-1)}$ représente la probabilité (conditionnelle) de rupture du k-ème élément dans le i-ème chemin de ruine, étant donné que les k-1 éléments précédents se dont déjà rompus.

Les inégalités suivantes :

$$P_{(-)i}^{(1)} < P_{(-)i}^{(2/1)} < \ldots\ldots < P_{(-)i}^{(N/1,\,2,\,\ldots,\,N-1)} \qquad (3)$$

sont évidemment respectées.

Dans le cas général, le calcul analytique de $P_{(+)}$ s'avère difficile. On se bornera ici à encadrer la probabilité globale de survie par deux nombres plus simples à calculer [5], [6] :

$$\prod_{i=1}^{N!} P_{(+)i} \leq P_{(+)} \leq \min_{i=1}^{N!} (P_{(+)i}) \qquad (4)$$

Il faut noter que la probabilité donnée par la borne inférieure se trouve du côté de la sécurité. Elle s'obtient en supposant que les N ! chemins sont statistiquement indépendants. Cette probabilité sera d'autant plus proche de $P_{(+)}$ que la corrélation entre les chemins de ruine sera faible.

En supposant les N éléments identiques et en imposant que la probabilité de survie du modèle $P_{(+)}$ soit au moins égale à une valeur de référence $P^{\circ}_{(+)}$ acceptée a priori, on peut écrire la relation :

$$P^{\circ}_{(+)} = (P_{(+)i})^{N!}$$

Une relation simple peut aussi être établie en ce qui concerne les probabilités de ruine :

$$P^{\circ}_{(-)} = 1 - P^{\circ}_{(+)} = 1 - (1 - P_{(-)i})^{N!} \simeq N!\ P_{(-)i}$$

3. BREVE COMPARAISON ENTRE LES MODELES SERIE, PARALLELE FRAGILE ET PARALLELE DUCTILE.

3.1. Pour illustrer cette comparaison, on considère l'exemple simple d'une poutre encastrée-appuyée soumise à une charge concentrée P (figure 5.a). La poutre possède deux sections potentiellement critiques 1 et 2.

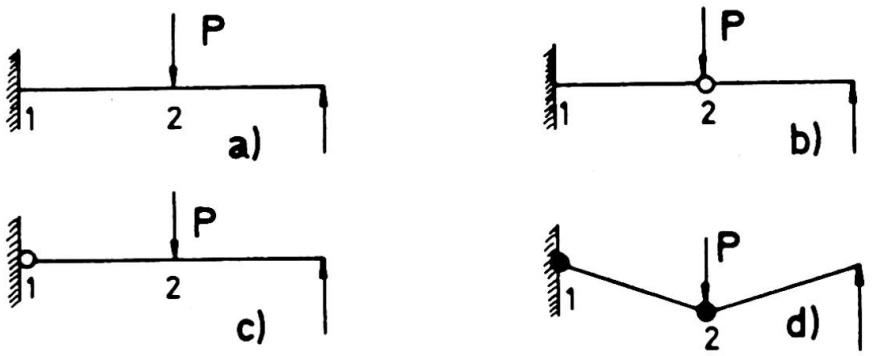

Figure 5

a) Lorsqu'on applique le concept du dimensionnement élastique, le modèle de comportement est, du point de vue probabiliste, un modèle série. Dans ce cas en effet, on considère que la structure est mise hors service quand on atteint la sollicitation maximum admissible dans la section 1 ou dans la section 2.

b) On considère à présent comme état-limite l'effondrement du système et on suppose que le matériau a un comportement du type fragile (sections à très faible ductilité). Dans ce cas, on peut définir deux chemins de ruine tout à fait distincts :

- la poutre cède d'abord en 2, mais en conservant une résistance suffisante à l'effort tranchant. La section 2 se transforme donc en une rotule (figure 5.b.). L'effondrement du système se produit quand la poutre cède dans la section d'encastrement ;
- la poutre cède d'abord en 1, qui devient une rotule libre (figure 5.c.). Le système se comporte alors comme une poutre biarticulée jusqu'au moment où il cède en 2, ce qui amène son effondrement.

Dans ce cas, le comportement du système peut être représenté par un modèle parallèle fragile.

c) S'il s'agit d'un matériau possédant une ductilité appréciable (par exemple une poutre en acier), la rupture se produit par formation du mécanisme de ruine de la figure 5.d.
Le modèle de comportement est alors le modèle parallèle ductile.

3.2. En ce qui concerne la relation entre la sécurité d'un élément et la sécurité d'ensemble, la figure 6 permet de comparer le coefficient η', correspondant au modèle série, avec le coefficient η'', correspondant au modèle parallèle ductile. Ils ont été définis par l'un des auteurs [6] comme le facteur par lequel il faut multiplier le coefficient de sécurité central d'un élément isolé pour que la probabilité de survie de la structure soit la même que celle de l'élément isolé.

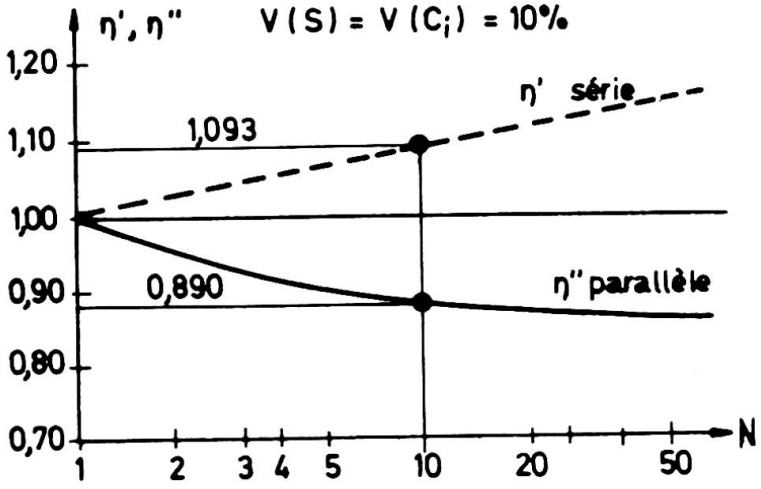

Figure 6

La comparaison est établie pour une probabilité de survie de la structure de $1\text{-}10^{-3}$ et une même distribution (logarithmo-normale) de l'action et des capacités portantes des éléments.

On considère par exemple le cas d'une association de N = 10 éléments identiques dont la capacité portante est caractérisée par un coefficient de variation V (C_i) = 10 %, qui doit supporter avec une sécurité de $1 - 10^{-3}$, une action S de coefficient de variation V(S) = 10 %. Si les 10 éléments sont disposés en série, le coefficient η' doit avoir la valeur 1.093. S'ils sont disposés en parallèle, η'' doit valoir 0.890.

Le tableau ci-dessous indique la valeur qu'il faut donner à la probabilité de survie d'un élément pour assurer à l'ensemble des 10 éléments une probabilité de survie de $1 - 10^{-3}$ dans les trois associations envisagées.

Modèle	série	parallèle fragile	parallèle ductile
$P_{(+)i}$	$1 - 10^{-4}$	$1 - 53.10^{-4}$	$1 - 120.10^{-4}$

3.3. Pour le modèle série qui schématise le comportement des constructions dont la ruine d'un élément amène la ruine d'ensemble, nous avons montré [1], [4], [6] que chaque élément doit avoir une probabilité de survie égale à $\sqrt[N]{P^{\circ}_{(+)}}$ (N = nombre d'éléments) pour que la probabilité de survie de la structure soit au moins égale à $P^{\circ}_{(+)}$.

Puisque pour n'importe quelle valeur de N> 2 l'inégalité

$$\sqrt[N]{P^{\circ}_{(+)}} < \sqrt[N!]{P^{\circ}_{(+)}} \qquad (5)$$

est vérifiée, on peut affirmer que :

Pour un même nombre d'éléments, N, et pour une même probabilité de survie du système, $P^{\circ}_{(+)}$, la sécurité d'un élément faisant partie d'un modèle série est toujours inférieure à la sécurité d'un chemin de ruine possible associé à un modèle parallèle fragile.

4. COMPORTEMENT DU TYPE MIXTE.

Beaucoup de structures se caractérisent par un comportement mixte (ductilo-fragile) [2]C'est le cas des constructions dont les sections ne possèdent qu'une capacité limitée de déformation plastique avant la ruine. Dans certaines circonstances, les deux types de comportement peuvent apparaître successivement. Par exemple, si plusieurs éléments mixtes (acier-béton) d'une structure sont soumis à un effort de traction, le béton épuise rapidement sa capacité de résistance en traction et se rompt (comportement fragile). L'effort est alors repris uniquement par l'acier dont le comportement est essentiellement ductile.

Une telle structure peut être schématisée par un modèle mixte présentant des éléments disposés à la fois en série et en parallèle (cfr. figure 4). L'évaluation exacte de la probabilité de survie s'avère, dans ce cas, pratiquement impossible. Néanmoins, on peut établir des bornes d'un intervalle qui encadre la valeur exacte de cette probabilité [4].

5. CONCLUSIONS.

Cette étude complète notre contribution au Rapport Préliminaire [1]. On analyse, du point de vue probabiliste, deux autres types de comportements : parallèle fragile et mixte.

La complexité du problème rend pratiquement impossible l'évaluation exacte de la sécurité,mais de tels modèles fournissent à l'ingénieur des indications précieuses pour l'évaluation de la sécurité des structures par rapport aux différents types de comportements structuraux.

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RESUME

On examine du point de vue probabiliste deux types de comportements structuraux: le comportement du type parallèle fragile et le comportement du type mixte. On présente une comparaison qualitative et quantitative entre les différents modèles.

ZUSAMMENFASSUNG

Man untersucht durch wahrscheinlichkeitstheoretische Betrachtungen zwei Typen von strukturellem Verhalten : das parallele spröde und das gemischte Verhalten. Ein qualitativer und quantitativer Vergleich der beiden Modelle wird angestellt.

SUMMARY

Two types of structural behaviour are considered from a probabilistic point of view: the parallel brittle and the mixed types of behaviour. Qualitative and quantitative comparisons are made between different models.

Considerations on Optimum Combination of Safety and Economy

Considérations sur le meilleur compromis entre sécurité et économie

Betrachtungen über den besten Kompromiss zwischen Sicherheit und Wirtschaftlichkeit

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The report |1| has served to emphasize that engineering design requires methods and concepts of probability. The process of selecting the probability of failure of a structure to achieve a stated objective (e.g. make minimum the total expected cost of the structure) is known as probabilistic optimum based-design |7|.

The goal of this discussion is to show that a few useful conclusions can be found in the safety-economy optimization process.

In order to evaluate the total expected cost of a structure C_t, the initial cost C_i, the cost of failure C_f and the probability of failure P_f are to be calculated first. Optimum probabilistic design requires the minimization of C_t :

$$C_t = C_i + C_f \cdot P_f \qquad (1)$$

The central safety factor of a structure is very nearly a linear function of $\text{colog}_{10} P_f$ |2|, |3| ,|6| :

$$\theta = \alpha_1 \cdot \text{colog}_{10} P_f + \alpha_2 , \qquad (2)$$

where α_1 is the increment of central safety factor required to reduce P_f by a factor of 10, and α_2 is a positive constant.

On the other hand, the initial cost C_i may be taken as a linear function of θ |2|, |5|, |8| :

$$C_i = \gamma_1 \cdot \theta + \gamma_2 = \beta_1 \cdot \text{colog}_{10} P_f + \beta_2, \qquad (3)$$

where γ_1, γ_2, $\beta_2 = \alpha_2 \gamma_1 + \gamma_2$ are constants coefficients and β_1 is the increment of the initial cost required to reduce P_f by a factor of 10.

Since the cost of failure C_f is generally independent of the initial cost,

equating the derivative of the total cost to zero gives the optimum probability of failure

$$P_{f_{op}} = \frac{\beta_1}{C_f . \ln 10} \tag{4}$$

and the optimum expected total cost

$$C_{t_{op}} = \beta_1 . \operatorname{colog}_{10} \frac{\beta_1}{C_f . \ln 10} + \frac{\beta_1}{\ln 10} + \beta_2 \tag{5}$$

Therefore the expression for the excess cost ΔC_t (overdesign or underdesign) is the following :

$$\Delta C_t = C_t - C_{t_{op}} = \beta_1 \left(\log_{10} \frac{P_{f_{op}}}{P_f} + \frac{1}{\ln 10} \frac{P_f - P_{f_{op}}}{P_{f_{op}}} \right). \tag{6}$$

Assuming $P_{f_{op}} = 10^{-4}$, a plot of $\Delta C_t / \beta_1$ as a function of P_f is shown in figure 1.

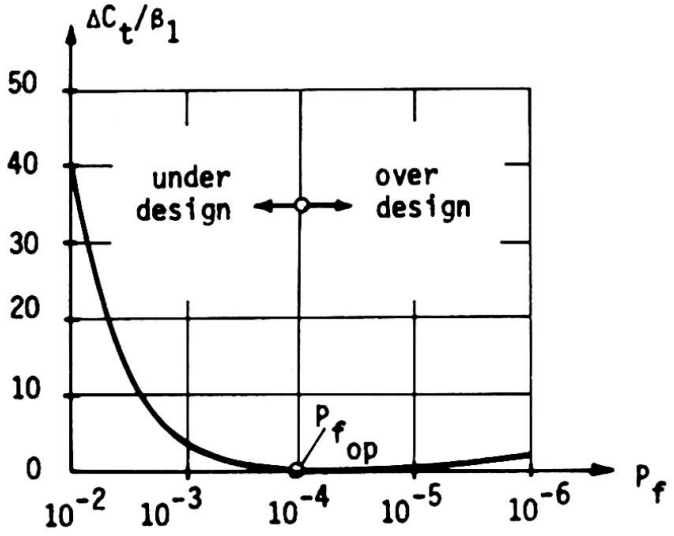

Figure 1 - Excess cost.

It then follows that :

- For the same absolute value of the safety difference

$$D = \operatorname{colog}_{10} P_f - \operatorname{colog}_{10} P_{f_{op}} \tag{7}$$

  the underdesign is more expensive than the overdesign ;
- In the region of overdesign, the excess cost is quite insensitive to the P_f variation ;
- In the region of underdesign, the excess cost is very sensitive to the P_f variation.

In the eventually that damage occurs, the loss C_f (costs of the building, potential loss of human lives and industrial damage caused by the failure) can be expressed as f times the cost C_i |5|.

The value of f gives a good indication of the magnitude of the damage that is caused by failure.

The optimum expected total cost (5) is a linear function of $\log_{10} f$ |4| :

$$C_{t_{op}} = \beta_1 . \log_{10} f + a \qquad (8)$$

where

$$a = \beta_1 . \operatorname{colog}_{10} \frac{\beta_1}{C_i . \ln 10} + \frac{\beta_1}{\ln 10} + \beta_2 .$$

A graph of $C_{t_{op}}$ as a function of $\log_{10} f$ is given in Figure 2.

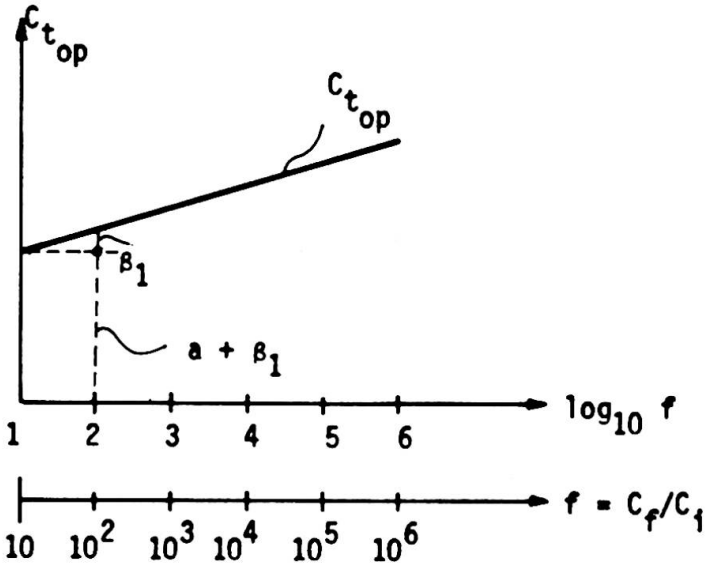

Figure 2 - Sensitivity of the optimal solution.

The foregoing considerations should permit a rational combination of safety and economy, in order to reach a satisfactory approach of an optimum solution.

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SUMMARY

Several independent attempts are made to evaluate an optimal value for the probability of failure and for the expected total cost. The sensitivity of the optimal solution to the magnitude of the damage that is caused by failure is also analysed.

RESUME

Plusieurs tentatives indépendantes sont faites pour obtenir la valeur optimale de la probabilité de ruine et du coût total attendu. La sensitivité de la solution optimale par rapport à l'importance du dommage causé par la ruine est également analysée.

ZUSAMMENFASSUNG

Verschiedene unabhängige Untersuchungen werden gemacht, um den optimalen Wert der Versagenswahrscheinlichkeit mit bezug auf die zu erwartenden Totalkosten festzulegen. Die Abhängigkeit der optimalen Lösung von der durch den Bruch verursachten Schadengrösse wird auch analysiert.

Results of the Application of Stochastic Programming for the Computation of Safety of Structures

Résultats de l'application de la programmation stochastique pour le calcul de la sécurité des structures

Ergebnisse aus der Anwendung der stochastischen Programmierung für die Berechnung der Sicherheit von Konstruktionen

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To the proposed application of stochastic programming for the computation of safety I want to make some additional remarks. My first point deals with the comparison of the well-known method of computing the probability of failure of structures on the basis of Failure Modes /1/ with the proposed concept. The second remark concerns the application of the proposed concept to the limit load analysis by nonlinear programming.

1. On the left hand side of figure 1 we have a formulation of the first limit load theorem as a linear programming problem. The solution of this primal problem gives us the maximum of the load factor which holds the equilibrium conditions as well as the linearized yield conditions, both combined in the matrix $\underline{B}$. In this special formulation the vector of primal variables $\underline{y}$ consists of the load factor y_1 and the redundant forces $y_2 \ldots y_{p+1}$; p is the redundancy of the structure.

On the right hand side of figure 1 we have a formulation of

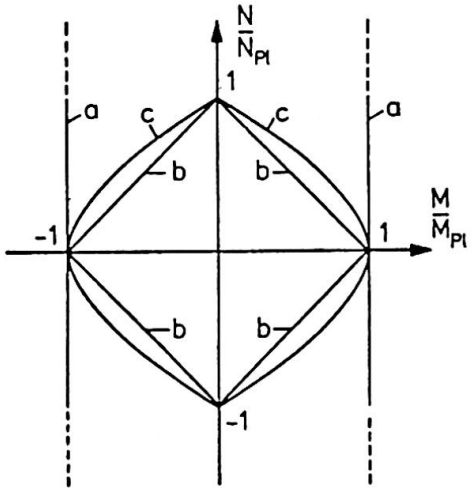
Primal Problem - First Limit Load Theorem	Dual Problem - Second Limit Load Theorem
maximize y_1	minimize $\underline{F}_o' \cdot \underline{z}$
subject to	subject to
$\underline{B} \cdot \underline{y} \leqq \underline{F}_o$	$\underline{B}_1' \cdot \underline{z} \geqq 1$
$\underline{y}_1 \geqq 0$	$\underline{B}_i' \cdot \underline{z} = 0, i=2 \ldots p+1$
	$\underline{z} \geqq 0$

$\underline{y}$: vector of primal variables
y_1: load factor, $y_2 \ldots y_{p+1}$: redundant forces
$\underline{z}$: vector of dual variables, i.e. strain velocities
$\underline{B}$: matrix, combining linearized yield conditions and equilibrium conditions
$\underline{F}_o$: vector of the right hand sides (e.g. fully plastic moments)
$\underline{F}_o\, z$: work of the internal forces

Figure 1: Linear Programming Problems for Limit Load Analysis

the second limit load theorem as a linear programming problem. The solution of this dual problem gives us the minimum of the work of the internal forces, i.e. the scalar product $\underline{F}^{!}\underline{z}$. The work of the external forces , given by $B_i^{!}\underline{z}$ must be not less unity. The equality constraints $B_i^{!}\underline{z}$ are the conditions of kinematic compatibility. B_i are columns of $\underline{B}$. Solving this dual problem is equivalent to finding the most critical out of all failure modes.

By the theorem of duality in linear programming we know that the load factor, that is the value of the objective function must be the same for both problems. Computing the probability of failure with the proposed concept then means just to look at the probability of failure by the most critical failure mode. Thus we have a lower bound for the probability of failure /1/. This is the reason why the sensitivity analysis becomes very important.



a) Failure Mode concept (Stevenson)

$$\frac{M}{M_{Pl}} \leqq 1$$

$$-\frac{M}{M_{Pl}} \leqq 1$$

b) linearized yield condition of proposed concept

$$\frac{M}{M_{Pl}} + \frac{N}{N_{Pl}} \leqq 1$$

$$\frac{M}{M_{Pl}} - \frac{N}{N_{Pl}} \leqq 1$$

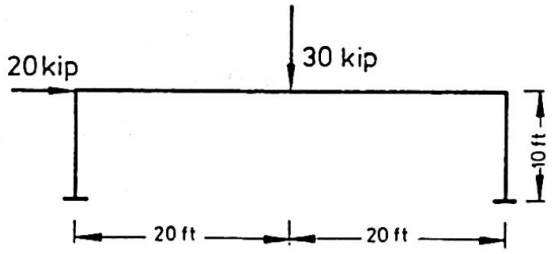
$$-\frac{M}{M_{Pl}} - \frac{N}{N_{Pl}} \leqq 1$$

$$-\frac{M}{M_{Pl}} + \frac{N}{N_{Pl}} \leqq 1$$

c) nonlinear yield condition

$$\frac{|M|}{M_{Pl}} + \left(\frac{N}{N_{Pl}}\right)^2 \leqq 1$$

Figure 2 : Yield Conditions of Beam Elements



coefficients of variation: 0.1

beam	column	Failure Mode concept		proposed concept	
mean M_{Pl}	mean M_{Pl}	load factor	p_f	load factor	p_f
200	100	1.0	0.56	0.99	0.54
320	50	1.0	0.63	0.98	0.59
450	75	1.5	$2.1 \cdot 10^{-4}$	1.46	$0.9 \cdot 10^{-4}$

Figure 3: Comparison of Different Computation Concepts

On the other hand in the computing of the probability of failure with regard of all possible failure modes the yield condition "a" in figure 2 is used. An interaction of normal forces and bending moments is either neglected, or the computation demands additional iteration. The lines "b" in figure 2 are bounding the feasible region of the proposed concept. The curves "c" belong to the quadratic function for the interaction of normal forces and bending moments.

For the comparison of the results which are given in figure 3 for Stevensons' test example /1/ we therefore must have in mind different yield conditions and different computation concepts.

2. The difference between the nonlinear yield condition and the linearized yield condition (figure 2) leads to the idea of computing the limit load by means of nonlinear programming. The nonlinear programming problem for limit load analysis is given in figure 4. An iterative method yields a solution of this problem. In the solution a number of yield conditions hold the equality sign. These nonlinear yield conditions are expanded into a Taylor series around the solution and around the stochastic variables $\underline{F}_o$. The nonlinear terms are suppressed and this results in a linear transformation of the stochastic variables $\underline{F}_o$ into the unknowns $\lambda, \underline{X}$. With such a linear transformation we can apply the described concept to the limit load analysis by nonlinear programming.

maximize λ

subject to

$$\Phi_i\ (\lambda, \underline{X}, F_{oi}) \geqq 0 \qquad i = 1 \ldots r$$

$$\lambda \geqq 0$$

λ : load factor

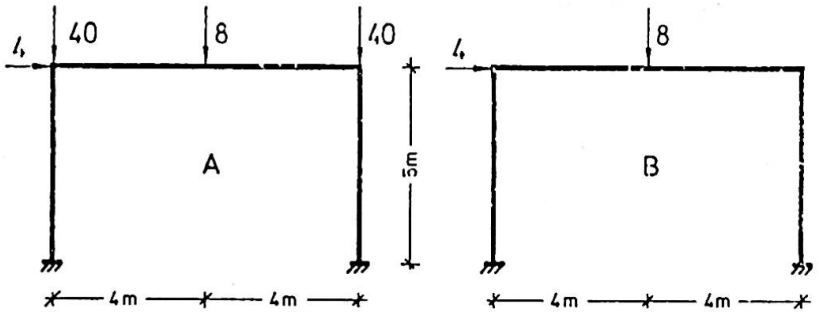
$\underline{X}$: vector of redundant forces

F_{oi} : plastic load bearing capacities (e.g. fully plastic moments)

Φ_i : nonlinear yield conditions

r : number of control points

Figure 4 : Nonlinear Programming Problem for Limit Load Analysis



M_{Pl} = 17.4 [Mpm] , N_{Pl} = 210.9 [Mp] in all sections

coefficients of variation : $c_v^P = 0.2$, $c_v^{M_{Pl}} = 0.1$

	system A		system B	
	linear	nonlinear	linear	nonlinear
load factor	1.56	1.81	1.93	2.00
p_f	$5.3 \cdot 10^{-3}$	$6.6 \cdot 10^{-5}$	$1.9 \cdot 10^{-5}$	$4.5 \cdot 10^{-6}$

Figure 5 : Comparison of Linear and Nonlinear Programming in Limit Load Analysis.

The results of linear and nonlinear programming in limit load analysis are compared in figure 5 with two different loading conditions. The differences in the probability of failure are caused mainly by the different load factors. The greater load factors of the nonlinear solution are due to the larger feasible region of the quadratic yield condition.

The design of structures for a given allowable probability of failure requires a solution of the above described problems in every step of iteration, i.e. computing the load factor and the probability of failure for given cross sections of the elements. For a single structure the computertime for a reliability-based design might cost too much. But if we look at problems like optimizing prefabricated elements for a large number of buildings or like the optimization of a building code for given restricted resources /2/ it is important to find structures with minimum cost which are safe. With the proposed concept I hope to have given a more general way of evaluating the safety of structures with regard to their mechanical properties.

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SUMMARY

Results obtained with the proposed concept for the computation of the probability of failure as it was described in the preliminary report are compared to results which are given by the well-known method of computing the probability of failure on the basis of failure modes. A second part of the paper deals with the application of the proposed concept to the limit load analysis by nonlinear programming.

RESUME

Les résultats de l'application de la programmation stochastique pour le calcul de la sécurité des structures - méthode présentée dans le Rapport Préliminaire- sont comparés avec les résultats donnés par le calcul de la probabilité de ruine usant la méthode des chaînes cinématiques . La seconde partie de l'exposé montre l'application de la méthode proposée au calcul à l'état limite, utilisant la programmation non-linéaire.

ZUSAMMENFASSUNG

Ergebnisse des im Vorbericht beschriebenen Verfahrens zur Berechnung der Versagenswahrscheinlichkeit von Konstruktionen werden mit der bekannten Methode, die Versagenswahrscheinlichkeit mit Hilfe der kinematischen Ketten zu bestimmen, verglichen. Der zweite Teil des Beitrags behandelt die Anwendung des vorgeschlagenen Verfahrens auf die Traglastberechnung mit nichlinearer Programmierung.

Improved Design Philosophy for the Structural System of Oil Storage Tanks

Un concept pour améliorer la structure porteuse des réservoirs à essence

Über verbesserte Entwurfsgrundlagen für das Tragsystem von Oeltanks

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1. Introduction

In December 1974, a large oil storage tank in a tank farm located in Western Japan collapsed due to cracks developed at the corner bottom plate. The collapse caused large amount of oil to flood into the sea, thereby giving serious damage to its biological environment. The accident urged re-examination of the safety of similar tanks. Under such circumstances, the authors' firm was asked to study and analyze the structural safety of large cylindrical tanks constructed on a deep reclaimed site in Yokohama City.

The investigation was aimed at: firstly, recommending measures to improve structural safety of tanks on the basis of fundamental considerations on them (Fig. 1) and comprehensive evaluation of the data obtained through the authors' observation of 18 emptied tanks which have sunk at a considerable degree; and secondly, analyzing static and dynamic behaviours of cylindrical oil tank constructed on deep, heterogeneous, loose soil easily subject to consolidation settlement, and establishing certain design criteria to be used as guidelines by the municipal authorities.

The present paper deals with comparative analyses of essential matters involved in "flexible-flexible" structures (Fig. 2) on a basis of the test findings and design improvement proposal derived from a series of analysis.

Fig. 1 Comparison of Present Modelling with Real Structure

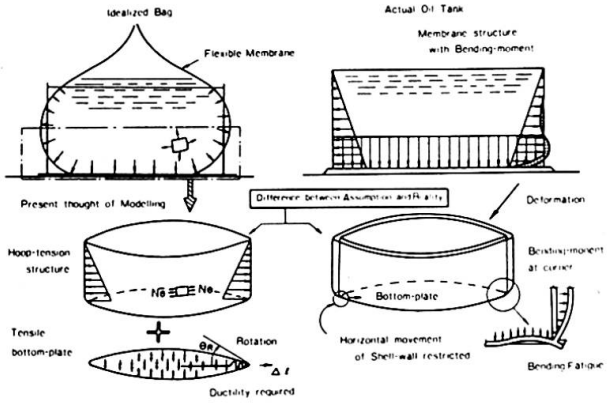
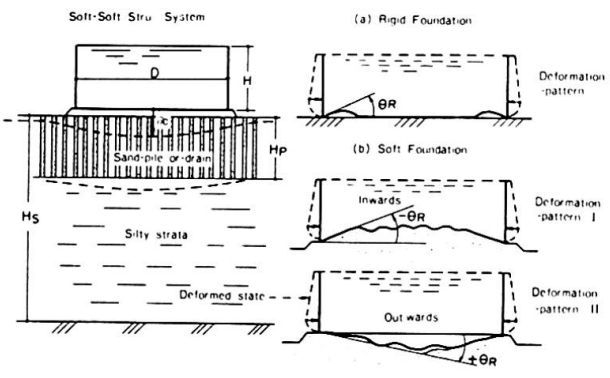


Fig. 2 Structural System and Bottom Plate Deformation Pattern



2. Behaviours of 18 Emptied Tanks

The tested tanks are grouped into three types, namely, 10,000 kℓ tanks (25 to 42 m diam. and 12 to 21 m high); 30,000 kℓ tanks (48 to 55 m diam. and 15 m high); and 63,000 kℓ (64 m diam. and 22 m high).

The tanks are underlain by the loose soil stratum of which volume compressibility coefficients (m_{vc}) range from 0.5 to 2 x 10^{-2} cm^2/kg in a depth of 20 to 50 m below grade. The soil is, in a range down to 7 to 23 m, stabilized by sand-drain method using 40 to 50 cm diam. pipes.

As compared with the levels recorded upon the completion of tanks, overall absolute settlements of tanks were 30 to 195 cm at circumferential places of the tank and differential settlements were 10 to 80 cm between the center and the edge. In some instances, mean residual deformation angles, overall settlements and differential settlements were in considerable agreement with those obtained from FEM analyses of a monolithic tank-soil model as shown in Table 1. By this, authors could have the confidence to proceed to detailed theoretical analysis.

The utmost emphasis in the checking was given to the corner at which shell and bottom plates are joined by fillet welding. As shown with statistical curves in Fig. 3 gained from processing measured data aquired at places of 2 m pitch along circumference of tanks, the plastic residual deformation angles of bottom plates at the corner due to the forced bending moment arising from filled or empty state of tank on plastically deformed supporting soil were found to be about two or three degrees on an average, though each measured value varied depending on the construction conditions. At some points where the underlying soil was very loose, an angle of as much as eight degrees was found.

Table 1 Tank-dimensions Measured and Theoretical values

	D (m)	H (m)	Clay Depth (m)	Foundation	Final Circum. Settlement (cm)	θ max	Number Inspected
A 10,000kL	25.00 ~ 42.53	12.20 ~ 21.00	8 ~ 48	Sand-pile and -drain	65 ~ 140	-174°	12
B 30,000kL	48.45 ~ 54.26	15.28	21 ~ 46	Preloading and Sand-drain	60 ~ 195	-173°	5
C 63,900kL	63.90	21.90	40	Vibro-floatation	180	-186°	1

* : from measured values

(Theory – corner rotation angle)

Solution Method	By FEM			By K-value Method		
	CASE 1	CASE 2	CASE 3	100kg/cm³	10 kg/cm³	1 kg/cm³
A (NO 24)	0.39°	1.00°	1.49°	0.85°	0.97°	1.02°
C (NO 3)	2.04°	2.50°	2.85°	2.11°	2.37°	2.53°

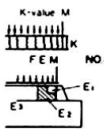
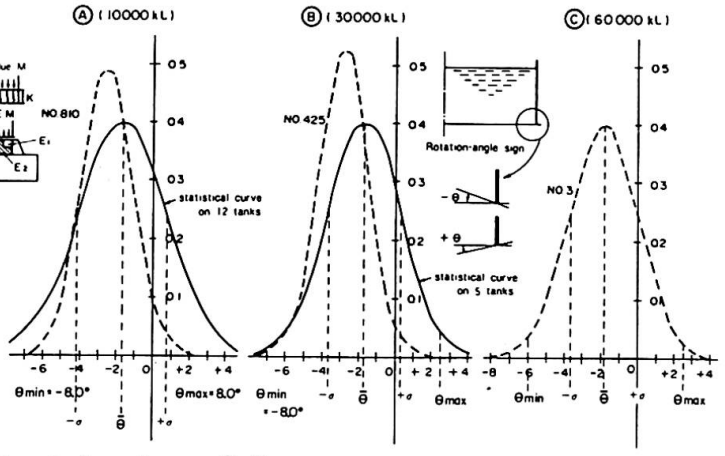


Fig. 3 Statistical Distribution of Rotated Angle



3. Theoretical Analyses of Tanks Constructed on Loose Soil

The tanks now under consideration are a kind of membrane structures subject to bending constraints at the shell-to-bottom joint. In addition, the tanks stand on loose soil whose properties remain unclarified by a quantitative analysis. Thus, it is theoretically difficult to make an accurate model that can properly represent these structures. For the purpose of present study, however, an attempt was made which could enable FEM-parametric

studies to be applied, regarding the tank and the soil as single structural body. As an important step, equivalent Young's moduli of soil easily subject to consolidation settlement were determined on the basis of a presumption that δ-ϵ curve properties of them could be regarded as a pseudo-elastic body.

By applying FEM to the semi-infinite elastic solid constituted by thick layers of loose soil with circular pressure loaded upon the surface, displacement and pressure distribution in the soil, and particularly the range of concentrated stresses in the soil located directly below the tank shell were obtained. Then, based on the stress-concentrated area thus determined, elasto-plastic transitional range for the tank and soil was established in a number of stages, and a local area at the shell-bottom junction was taken out and given parametric studies to see the behaviours in the plastic range as a single structural body. Thus obtained are parametric chaning conditions of bending deformation and stresses in the bottom plate at the junction and in the soil as illustrated in Table 2.

Table 2 Parametric Analysis Results by FEM with Corner Portion

Combination of Youngs Modulus of Soil

	E_1	E_2	E_3
CASE 1	130	130	130
CASE 2	13	65	130
CASE 3	3	13	65

Mechanical State of Corner Botm Plate

	σ_B	δ_v (mm)	P_B	θ_B (°)
CASE 1	5.75 (-4.46)	9.60	4.20	2.04
CASE 2	2.14 (-10.67)	1.15	1.32	2.50
CASE 3	1.00 (-13.26)	1.46	0.61	2.85

Thereafter, in order to assist in considerations about detail improvements and formulation of practical calculation method, chaning conditions of the bending moments and deformation of the junction were ascertained by giving parametric changes to tank-supporting soil spring while making reference to soil's subgrade reaction coefficients as found experimentally in the existing compacted soil foundation as shown in Table 3.

Table 3 Mechanical State Values on Improved Corner Details

		K = 1000 kg/cm³	K = 100	K = 10	K = 1
Improved(1)	δ_v (mm)	0.435 (0.755)	1.75 (1.95)	5.7 (5.71)	27.3 (30.4)
	P_B (kg/cm²)	43.50 (75.50)	17.50 (19.50)	5.70 (5.71)	2.73 (3.04)
	M_B (t m/m)	3.25 (2.06)	3.51 (1.30)	3.09 (0.80)	2.93 (0.48)
	σ_B (t/cm²)	2.53 (8.66)	2.74 (5.50)	2.40 (3.40)	2.28 (2.07)
Improved(2)	δ_v (mm)	0.62	2.10	7.51	44.7
	P_B (kg/cm²)	62.40	21.00	7.51	4.47
	M_B (t m/m)	3.75	3.22	2.62	1.82
	σ_B (t/cm²)	2.91	2.48	1.71	0.97

Tank Scale 63,000 kL D = 63.90 m H = 21.90 m

Inferred from these analytical results, three types of hysteresis loop on bending moment to curvature corresponding to the kind of combination of the mechanical state of corner bottom plate with the degree of deformation characteristics of its foundation are thought to exist as illustrated in Fig. 4.

Fig. 4 Moment-Curvature Curves for Corner Bottom Plate

From various considerations on them the type ③ may be said much more better and safer even for bending fatigue in reference to fracture mechanics.

In order to see dynamic behaviours of the assumed tank-soil structure during earthquakes, response analyses of liquid contained in a rigid cylinder were made using ground surface waves in which long natural components are predominant as measured for reclaimed loose land and a few kinds of other ordinary strong waves. Fig. 5(a) shows a typical liquid response to the Niigata Earthquake-wave in 1964 which has the characteristic of transfer to long period of wave from usual earthquake pattern after occurrence of soil liquefaction, representing the overflow type due to large liquid-surface displacement with the value of 1.7 m shown by sign (D). Fig. 5(b) shows a typical response to the severe Kwanto Earthquake-wave on Sept. 1, 1923 restored by Prof. N. Nasu which has the maximum acceleration of 390 gals in the usual strong earthquake pattern, making characteristic of occurrence of very high pressure to shell-wall and bottom plate with the value of 0.8 Kg/cm^2 shown by sign (P) which forms an important condition for check against the fracture of corner bottom plate at any strong earthquake. With these response analyses the conditions of liquid agitation, the acting pressure distribution on tanks and the deformation of liquid were obtained.

Fig. 5(a) Sloshing Response of 63000kℓ Tank (1)

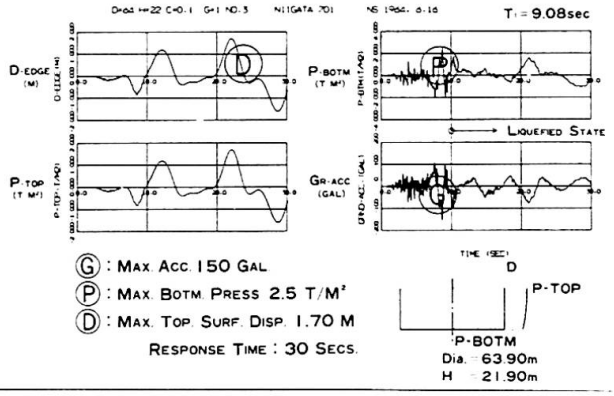
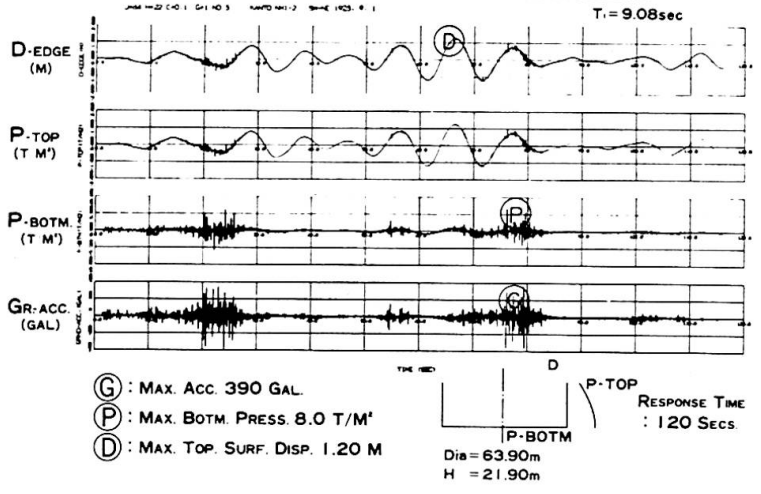
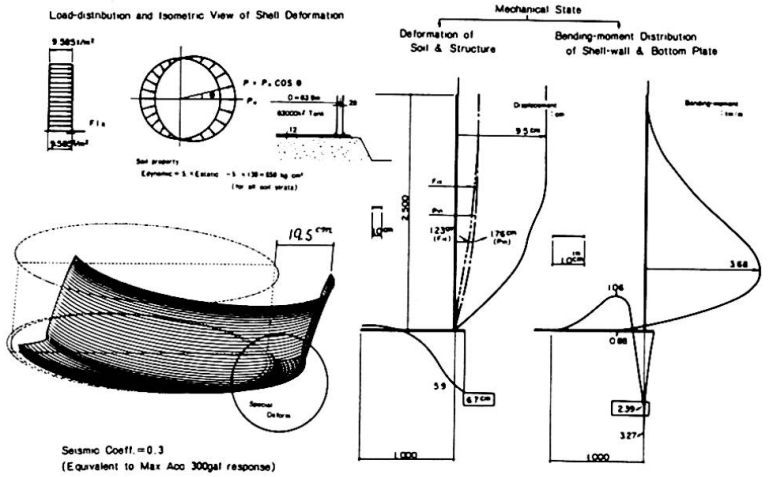


Fig. 5(b) Sloshing Response of 63000kℓ Tank (2)



Taking into considerations of the above sloshing effects and using the distribution pattern of active pressure calculated by the seismic coefficient of 0.3 prescribed simply in Japan, asymmetrical aspects of horizontal force acting on the assumed tank-soil structure were elucidated by FEM. From these results, it was found that compressive force, bending moment and special deformation at the tank lower end would possibly cause buckling of the shell plate as shown in Fig. 6.

Fig. 6 FEM Analysis on Shell-Soil Str. System for Horizontal Pressure



On the other hand, basic dynamic analyses by both frequency-analysis-method and earthquake response of tanks were conducted on the rocking and sway motions, regarding the tank as a rigid body supported by vertical and horizontal springs, and sinking of the shell into the foundation in various cases was measured as shown in Fig. 7. The results calculated on the

level of response to earthquake acceleration of 300 gals show that such sinking generally resembles one caused by asymmetrical horizontal force; therefore, large sinking thus caused was considered to lead to large deformation and in some worse cases failure or fracture of the bottom plate.

4. Improvement in Design

From the foregoing analyses, the following were considered a prime consideration in improving the structural safety of large oil tanks constructed on reclaimed land consisting of deep, loose soil in view of preventing dreadful pollution or danger inevitably accompanying huge compensation.

(1) Compaction methods for loose silt layers should be improved to ensure more effective and economical compaction than is now available. In design practice, the initial consolidation settlement should be standardized and compiled with at higher values than are now commonly adopted as shown in Fig. 8.

(2) As for the foundation, appropriate detail design should be adopted that will gradually change and decrease, in a number of stages, the equivalent Young's modulus especially for a certain depth and width of foundation located directly below the shell as illustrated in Fig. 9 and 10.

(3) In combination with such foundation detail, appropriate detail design in shape and weld should be adopted that will ensure the mechanical safety of shell-to-bottom junction against repetitive stresses and fatigue failure as illustrated in Fig. 9 and 10.

(4) To prevent earthquake damage, appropriate countermeasures should be devised against the liquefaction failure of the ground and the failure of parts of foundation on which the shell-bottom junction is located as illustrated in Fig. 9 and 10.

Fig. 7 Diagram of Vertical Displacement of Corner Portion at Earthquake State

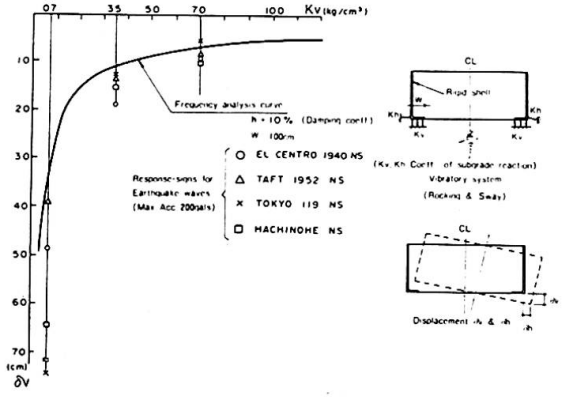


Fig. 8 Recommended Degree of Preliminary Consolidation

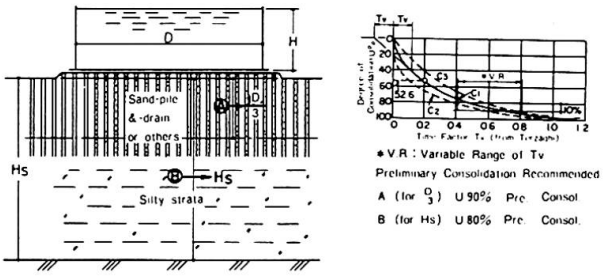


Fig. 9 Example of Improved Detail(1) around the Corner Portion

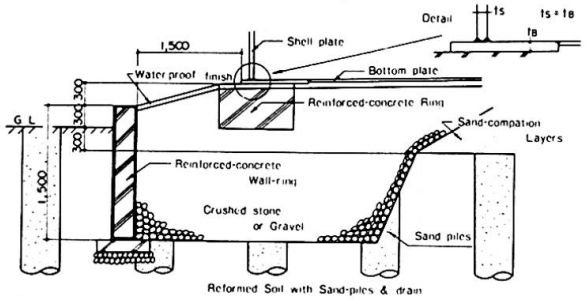
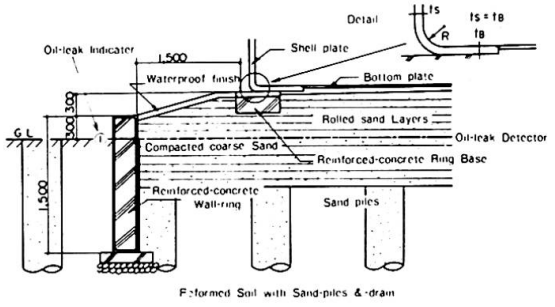


Fig. 10 Example of Improved Detail (2) around the Corner Portion



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SUMMARY

A large tank built on a deep and clayish soil should be a complete flexible-flexible structural system of a pure membrane structure and a soft soil. However, the tank designed by the current standards has unavoidable bending constraints at its corners. The authors have investigated eighteen large tanks, and have compared these with static and dynamic analyses of their flexible-flexible structural system. They propose a concept for improving the design of the tank-soil structural system.

RESUME

Un grand réservoir bâti sur un terrain argileux devrait être un système structural flexible composé d'une membrane et d'une terre molle. Cependant, les réservoirs construits selon les normes courantes présentent inévitablement des contraintes de flexion aux coins. Les auteurs ont fait des mesures sur dix-huit grands réservoirs et ont comparé celles-ci avec les calculs statiques et dynamiques de leur système structural flexible. Ils proposent un concept pour améliorer le projet du système structural réservoir-terre.

ZUSAMMENFASSUNG

Ein auf Tonboden gebauter Oeltank sollte ein flexibles Tragsystem aus einer Membrane und einem weichen Boden darstellen. Der nach den gewöhnlichen Normen gebaute Tank wird aber unvermeidlich Biegespannungen in den Ecken aufweisen. Von den Verfassern wurde die Nachprüfung von achtzehn Behältern durchgeführt, und mit statischen als auch dynamischen, numerischen Untersuchungen verglichen. Verbesserte Entwurfsgrundlagen werden für das Tragwerksystem Tank-Boden vorgeschlagen.

Die Ermittlung der mittragenden Breite im Stützenbereich von Durchlaufträgern

Determination of the Effective Width of Continuous Girders in the Column Zone

Détermination de la bande porteuse des poutres continues dans la région des appuis

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Zur Ermittlung des Spannungszustandes von Biegeträgern mit breiten Gurtungen bedient man sich häufig einer mittragenden Breite, um nach den üblichen Methoden der Festigkeitslehre rechnen zu können.

Diese mittragende, ideelle Breite berücksichtigt die Schubverformungen des als Scheibe zu betrachtenden Gurtes und den dabei auftretenden Gurtspannungsverlauf innerhalb des untersuchten Querschnittes.

Zur Bestimmung der mittragenden Breite gibt es eine Reihe bekannter Verfahren, die alle die Darstellung der Momentenzustandslinie als Fourierreihe für die Bestimmung der Scheibenspannungen im Gurt zur Grundlage haben.

Für die Belastung eines Einfeldträgers durch eine konstante Streckenlast oder eine mittige Einzellast und den daraus resultierenden parabolischen bzw. dreieckigen Momentenverlauf läßt sich die mittragende Breite z. B. für den Ort des maximalen Momentes ermitteln und für die praktische Handhabung als Kurve über dem Verhältnis Gurtbreite zu Spannweite auftragen (Bild 1). Dabei zeigt sich, daß die Konvergenz der Fourierentwicklung beim dreieckigen Momentenverlauf (Bild 1b) wesentlich schlechter ist als beim parabolischen (Bild 1a).

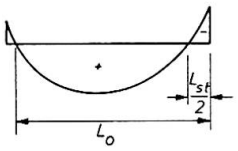
Die Ermittlung der mittragenden Breite gestaltet sich jedoch für den Stützenbereich von Durchlaufträgern weitaus schwieriger, weil der Momentenverlauf sehr unterschiedlich sein kann und die Konvergenz noch schlechter ist. Im Bereich des Stützenmomentes kann die Momentenfunktion allgemein durch

$$M(x) = Ax + Bx^2 \quad (0 = x = L_{st}/2)$$

ausgedrückt werden. Dabei liegt der Momentenverlauf beliebig zwischen den Randwerten B=0 (dreieckiger Verlauf) und A=0, B=q/2 (Hohlparabel, Kragarm durch Streckenlast belastet).

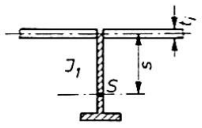
Somit wird die Darstellung der mittragenden Breite als Kurve über dem Verhältnis Gurtbreite zu Spannweite praktisch nicht möglich.

Üblicherweise wurde deshalb der negative Momentenbereich von Durchlaufträgern als Dreieck mit gleichem Maximalwert idealisiert und dafür die mittragende Breite berechnet. Als Länge L_{st} dieser Momentenlinie wurde dabei der Abstand der Momentennullpunkte verwendet (Bild 2).



Kurvenparameter

$$\delta' = \frac{2L_o}{L_{st}}$$



Querschnittskenngröße

$$\gamma = \frac{t_i \cdot L_{st} \cdot s}{J_1}$$

Aus Gründen der Sicherheit einerseits und der Wirtschaftlichkeit andererseits ist es notwendig, ein möglichst genaues Verfahren zu verwenden und gleichzeitig den Momentenverlauf möglichst exakt zu beschreiben.

Kehren wir zum betrachteten Stützenbereich zurück, so führt die Erfassung des tatsächlichen Momentenverlaufes gegenüber dem dreieckigen zu einer kleineren mittragenden Breite. Demnach ist aus Gründen der Sicherheit die übliche Idealisierung nicht zu empfehlen.

Die Vergleichsrechnung eines Straßenbrückenhaupträgers über drei Felder mit je 22,5 m Spannweite hat ergeben, daß sich bei Berücksichtigung einer dreieckigen Momentenlinie die mittragende Breite über einer Innenstütze gegenüber dem genauen Wert um 22% zu groß ergibt! Bei unendlich breiten Gurten wäre der Wert sogar um 33% größer!

Am Wissenschaftsgebiet Metallbau der Technischen Universität Dresden wurden in Vorbereitung der Überarbeitung unserer Stahlbrückenbauvorschriften Untersuchungen angestellt, um ein einfaches, rasch handhabbares Verfahren zur exakteren Bestimmung der mittragenden Breite zu erhalten.

Als mögliche Lösung wird nun vorgeschlagen, die Länge des Stützmomentenbereiches L_{st} mittels eines Korrekturfaktors α derart zu verkürzen, daß eine dreieckige Momentenlinie der Länge $L' = \alpha \cdot L_{st}$ die genaue mittragende Breite liefert.

Ausgehend von der Charakteristik des Momentenverlaufes $\delta = 2\,L_o/L_{st}$ und einer Querschnittskennzahl des den Gurt unterstützenden Steges γ (Bild 2) läßt sich der Faktor α angeben. Für $\delta = \infty$ erhält man einen geradlinigen Stützenmomentenverlauf (also ein Dreieck), damit wird $\alpha = 1{,}0$. Für $\delta = 1$ ergibt sich die Momentenlinie eines durch eine Streckenlast beanspruchten Kragarmes und α wird zu einem Minimum. In Bild 3 sind die α-Werte für alle denkbaren Stützenmomentenverläufe aufgetragen. Dabei liegt α zwischen 0,58 und 1,0. Die gezeigte Kurventafel gilt exakt nur für einen unendlich breiten Gurt.

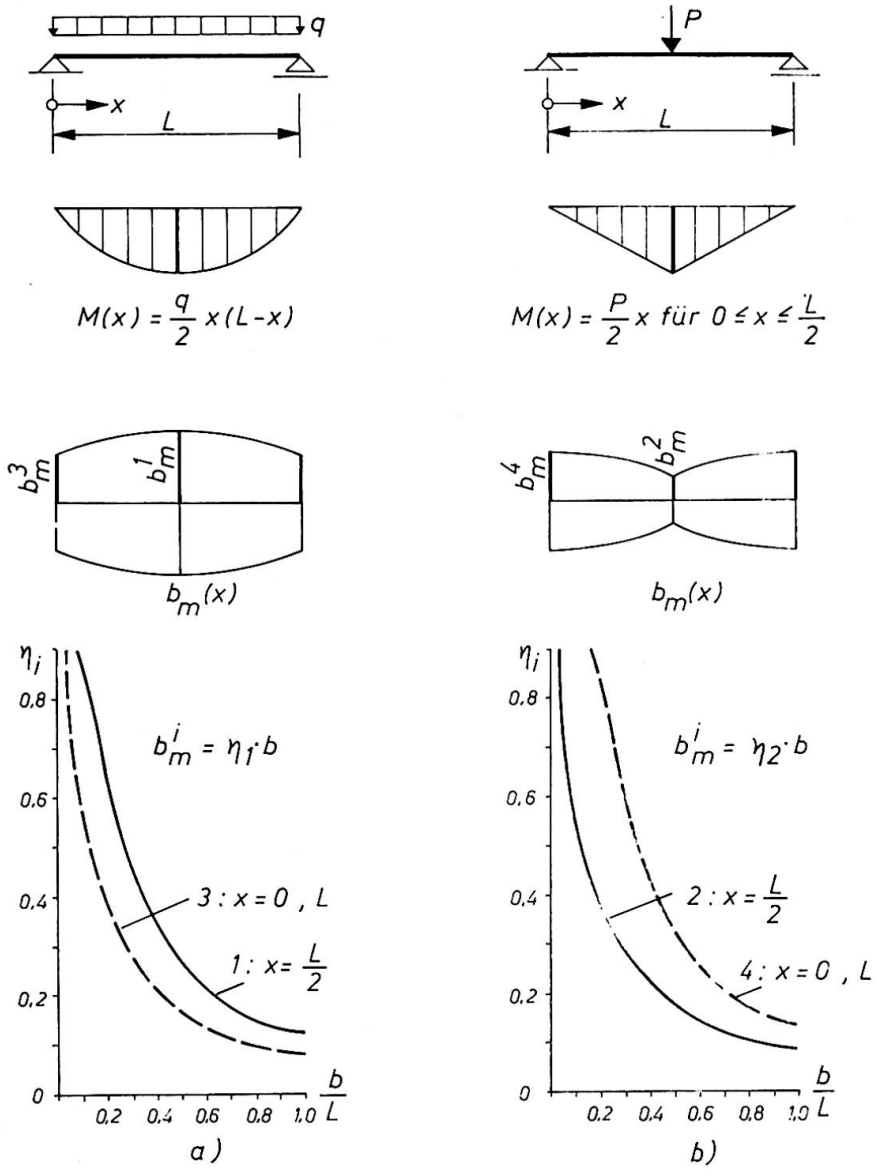


Bild 1 : Mittragende Breite a) bei Streckenlastbeanspruchung
b) bei Einzellastbeanspruchung

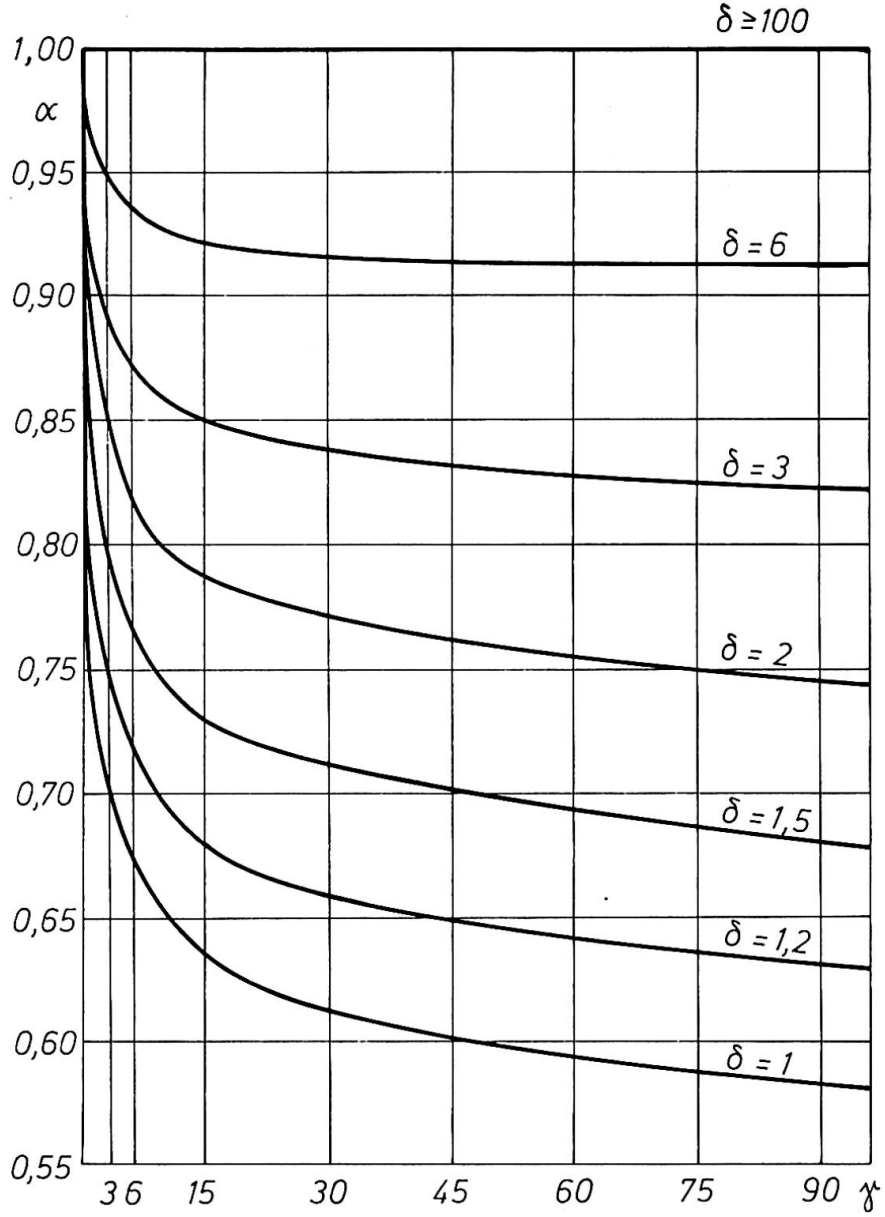


Bild 3 : Korrekturfaktoren α

Wendet man die Korrekturwerte auf die praktisch vorkommenden endlich breiten Gurte an, so erhält man mittragende Breiten, die kleiner sind als die tatsächlichen exakten Werte. Das Verfahren liefert also dann auf der sicheren Seite liegende Näherungswerte, deren Abweichung vom Sollwert geringer ist als bei der Dreieckmethode.

Für das oben zitierte Beispiel beträgt die Abweichung rund 10%.

Es laufen weitere Arbeiten, in deren Ergebnis exakte Werte für andere Randbedingungen der Gurte erhalten werden sollen.

Über die Kenntnis des Korrekturfaktors α läßt sich nun sehr rasch die exakte mittragende Breite bei Benutzung der Werte für eine dreieckige Momentenlinie ermitteln.

Die gezeigte Vorgehensweise gestattet damit eine genauere Erfassung der Tragsicherheit im Stützenbereich, ohne daß der Projektierungsaufwand spürbar erhöht wird.

ZUSAMMENFASSUNG

Zur Bestimmung der mittragenden Breite im Stützenbereich von Durchlaufträgern geht man meist von einer dreieckförmigen Idealisierung der Momentenlinie aus. Dadurch werden auf der unsicheren Seite liegende Ergebnisse erhalten. Der Beitrag zeigt eine Möglichkeit, über eine korrigierte Länge dieser dreieckigen Momentenlinie genauere und sicherere Werte ohne einen spürbaren Berechnungsmehraufwand zu erhalten.

SUMMARY

For determining the effective width of continuous girders in the column zone, mostly one starts from a triangular idealization of the moment curve. The values obtained are consequently on the uncertain side. This contribution shows one way for obtaining more precise and safe values by means of a corrected length of this triangular moment curve without any increase in the calculation.

RESUME

Pour déterminer la bande porteuse de poutres continues dans la région des appuis, on part le plus souvent d'une idéalisation triangulaire de la ligne des moments. On obtient ainsi des résultats se trouvant du côté incertain. L'article montre une possibilité d'arriver à des valeurs plus précises et plus sûres en partant d'une longueur corrigée de cette ligne des moments "triangulaire" sans une augmentation sensible des calculs.

Model Contribution to the Design and Safety Control of Large Structures

Contribution des modèles physiques au projet et au contrôle de la sécurité des grandes structures

Beitrag des Modellversuchs an den Entwurf und die Sicherheitskontrolle von grossen Bauwerken

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1. SAFETY ANALYSIS (ON A MODEL) AND OPTIMIZATION

The model study can be conducted as:

I) a modern method of stress analysis;

II) a tool evaluating the critical, or ultimate load.

In case I) the model, usually associated with an electronic processing device, operates as a clever stress calculating machine, and the results obtained can be compared with those provided by using computers of great capacity.

In case II) the ultimate carrying capacity of the structure can be evaluated and the investigation can also concern various types of loads. In this case, the study is completed by determining the "minimum" overall factor of safety of the structure.

2. ELASTIC MODELS

Widely used in stress analysis, they can be subdivided in two groups:

- the first group concerns models of plane structures; the importance of these methods has decreased owing to the ever greater use of the "finite element method".
- The second group generally deals with three-dimensional structures.

In statical tests, electrical deflectometers and extensometers, usually applied to the surface of the model in various directions, measure displacements and strains respectively (fig. 1). The model material may differ from the prototype material, providing it obeys Hooke' s law and its Poisson' s ratio differs slightly.

The model operates as an "analogical computer", and the final results obtained, easily collected by means of a computer, can be compared with the theoretical ones.

In dynamic tests electromagnetic exciters are applied and piezoelectric accelerometers are used as measuring instruments. The strains are still furnished by strain gauges connected to regular amplifying and recording electrical equipment.

Auxiliary masses are conveniently distributed throughout the model to comply with dynamic similitude (fig. 2).

3. STRUCTURAL MODELS

These serve for testing beyond the elastic range and are usually made of the same materials as the prototype. This is possible for steel and reinforced or prestressed concrete structures when suitable scale (1:5 - 1:20) models are used (fig. 3). But for large structures, such as dams, reasons of economy make it necessary to adopt small scale models (1:50 - 1:150), and then model materials whose mechanical properties are "reduced" with respect to those of concrete, in accordance with similitude.

For these models, ISMES has long been using microconcrete simulating the properties of concrete up to failure, and adopting the technique of "wet" models (with waterproof coating) that are pratically without internal stresses.

The tests are conducted in two successive stages. In the first stage, "at working load", the model strains are investigated, under loadings corresponding to the actual working loads.

The second stage concerns "ultimate load tests", and it is made by gradually increasing the applied loads. The ratio between the maximum load actually supported before the collapse and the normal working one is assumed as the overall "factor of safety" of the structure with respect to that "type" of load.

Special mention should be made of earthquake simulation in dynamic tests. ISMES has long had the necessary equipment, which also fully meets all the requirements (fig. 4).

4. GEOMECHANICAL MODELS

These are a speciality of ISMES and investigate structures resting on foundation whose equilibrum or settlement conditions may affect the safety of the structures themselves (large bridges, dams, power or highway tunnels, etc).

5. ON STRUCTURAL SAFETY IN THE DESIGN STAGE

Models are of great importance during the design of large structures, especially if they are structurally complicated and highly hyperstatic.

Of the many cases studied at ISMES, mention is made for:

- the tests carried out on elastic models of large viaducts such as, for instance, the Polcevera and Maracaibo bridge type; or of tall buildings, such as the reinforced concrete building in Montreal, Canada (fig. 5);
- the analysis, by means of structural models carried to failure, of new types of structures. Of the unusual cases mention will be made of the models of prestressed concrete vessels for nuclear reactors and of those of St. Mary' s Cathedral in San Francisco, California (fig. 6);

the geomechanical models used to analyze several concrete dams modeled with their foundation, e. g. the Kurobe IV dam (in Japan) and to investigate the stability of power or highway tunnels.

7. ON THE SAFETY FACTOR OF EXISTING STRUCTURES

Models can be of great value when the stability and safety degree of existing structures are to be checked. Particularly when large structures are to be verified, especially when actual statical or dynamic (seismic) conditions were not entirely foreseen in the design stage.

Among the models which yelded highly significant results for evaluating the safety factors brief mention will be made of:

- the failure tests on 1:4 scale models (fig. 7) to investigate the safety of the main columns carrying the "Duomo", i. e. the Cathedral in Milan. The two different materials used (Candoglia marble and Serizzo granite) and the geometry of the individual blocks were identical with those of the prototype.
  The stress conditions in the masonery dome bearing the main spire of the Cathedral were also examined on a large 1:15 scale elastic model (fig. 8).
- Sub-horizontal microcracks were found on the upstream of a large Italian arch-gravity dam. The influence of these cracks on the safety was investigated on a large structural model in which the microcraks had faithfully been reproduced.
- The effect of the foundation rock anysotropy horizontally stratified on a gravity dam in Spain was studied on geomechanical models (fig. 9).

REFERENCES: For more details on testing and results, please ask for Technical Bulletins of ISMES - P. O. Box 208, 24100 Bergamo (Italy).

SUMMARY

The present-day contribution of physical models to the design and safety control of large structures are presented; such investigations are systematically carried out at the author's laboratories.

RESUME

La contribution actuelle des modèles physiques au projet et au contrôle de la sécurité des grandes structures est présentée; de telles recherches sont effectuées de façon systématique aux laboratoires de l'auteur.

ZUSAMMENFASSUNG

Der heutige Beitrag des Modellversuchs an den Entwurf und an die Sicherheitskontrolle von grösseren Tragwerken wird dargestellt. Solche Untersuchungen werden systematisch in der Versuchsanstalt des Autors durchgeführt.

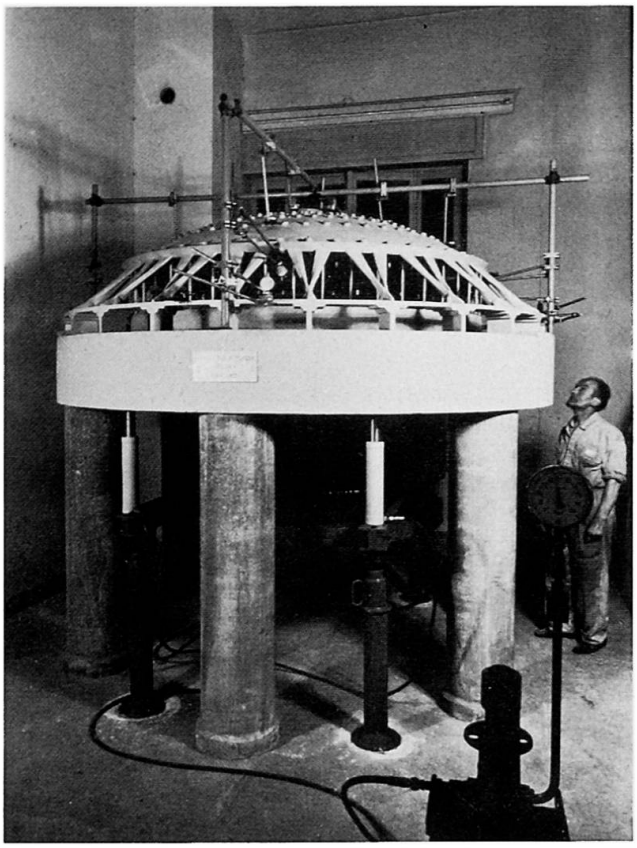

FIG. 1 1:50 scale elastic model of the 130 m dia. circular reinforced concrete shell roof of Norfolk Cultural Center (Virginia, USA). Statical tests.

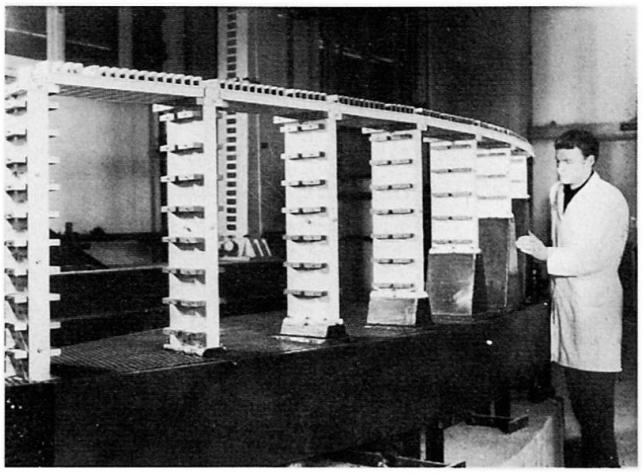

FIG. 2 1:100 scale elastic model of the curved highway viaduct across the Lao river (Calabria, Italy). Maximum height of viaduct piers 80 m. Dynamic tests.

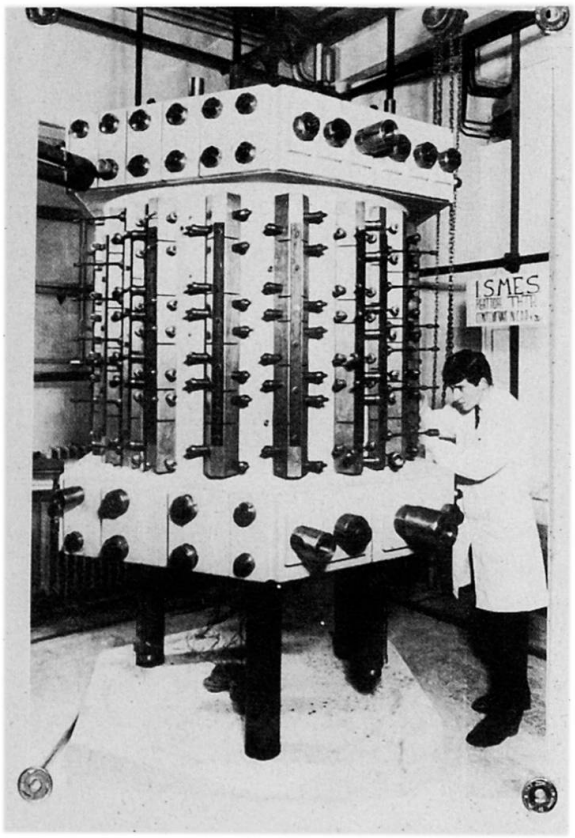

FIG. 3 1:20 scale structural model of the prestressed concrete vessel of the THTR nuclear reactor.

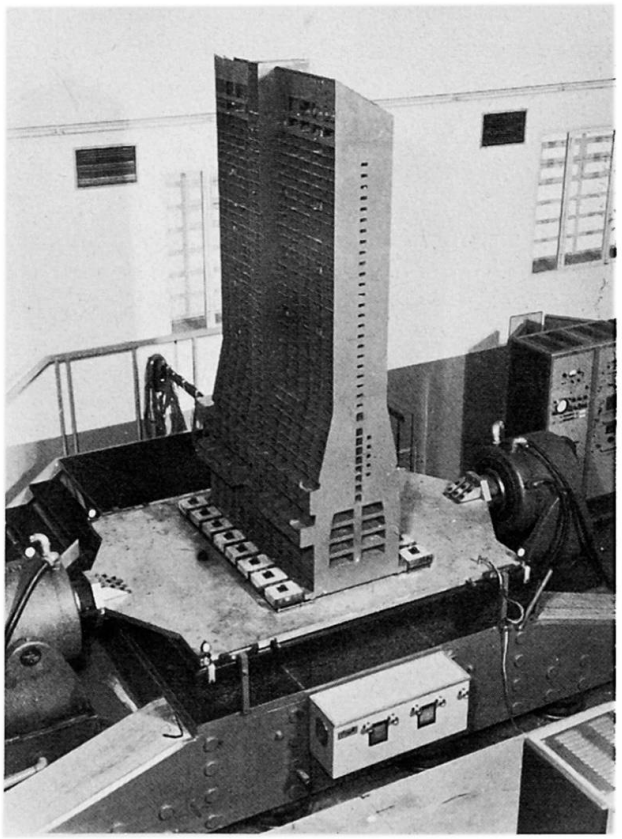

FIG. 4 1:40 scale elastic resin model of Parque Central Skyscraper, Caracas. Dynamic tests taking into account the foundation soil-structure interaction.

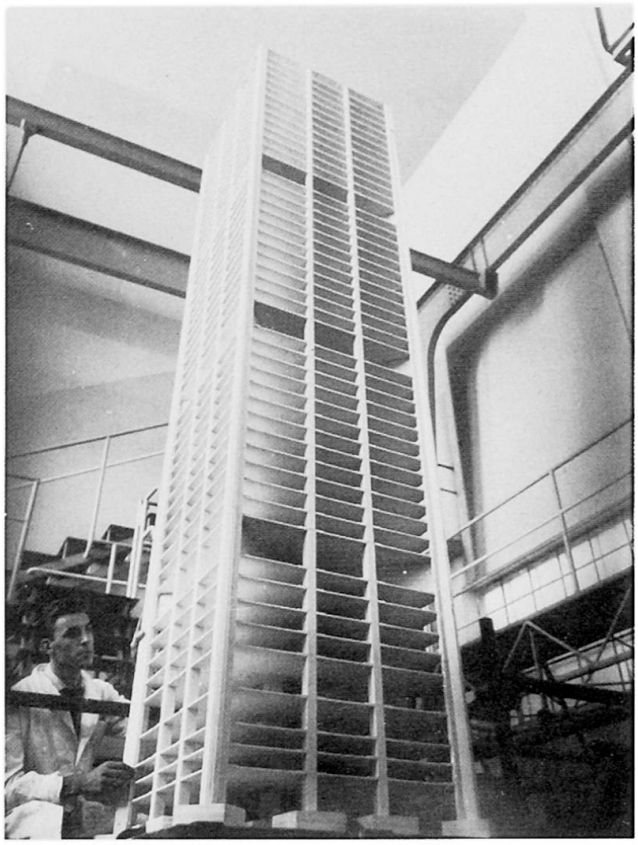

FIG. 5 1:52 scale celluloid elastic model of the 145 m high reinforced concrete building of Place Victoria in Montreal, Canada. Dynamic (seismic) tests.

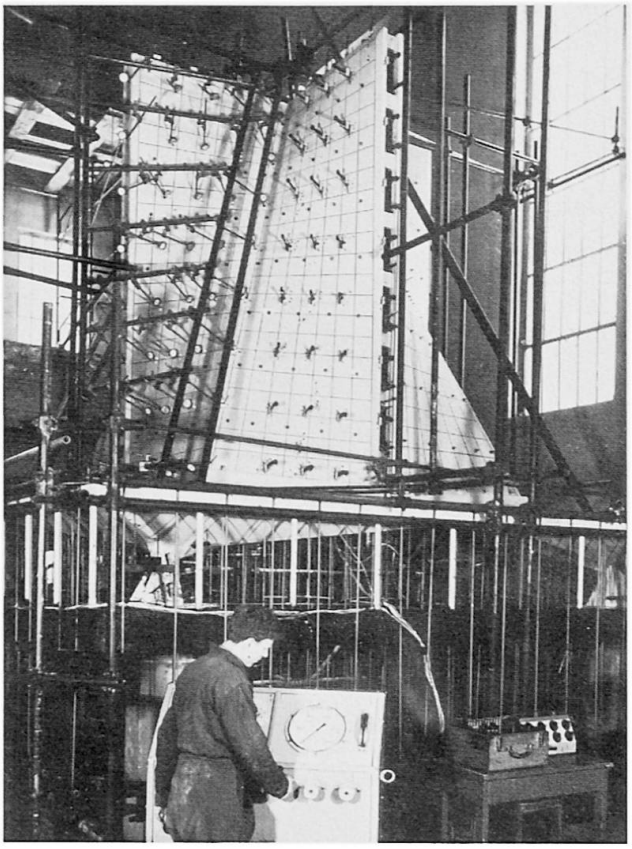

FIG. 6 1:15 scale structural model of the new Cathedral in San Francisco, California, USA, designed by P. L. Nervi. The model is ready for tests to failure.

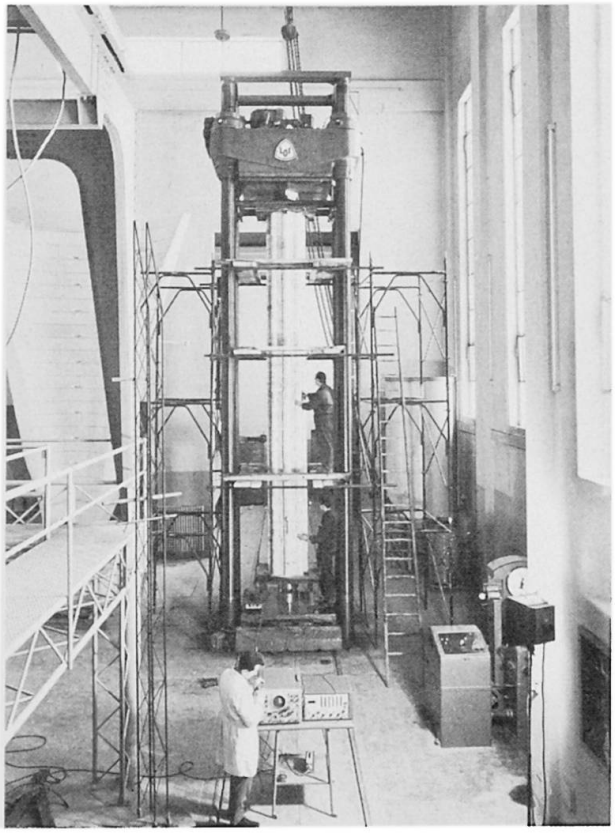

FIG. 7 1:4,7 scale structural model of the main columns carrying the masonry dome of the Cathedral in Milan. The model is ready for the axial compression test.

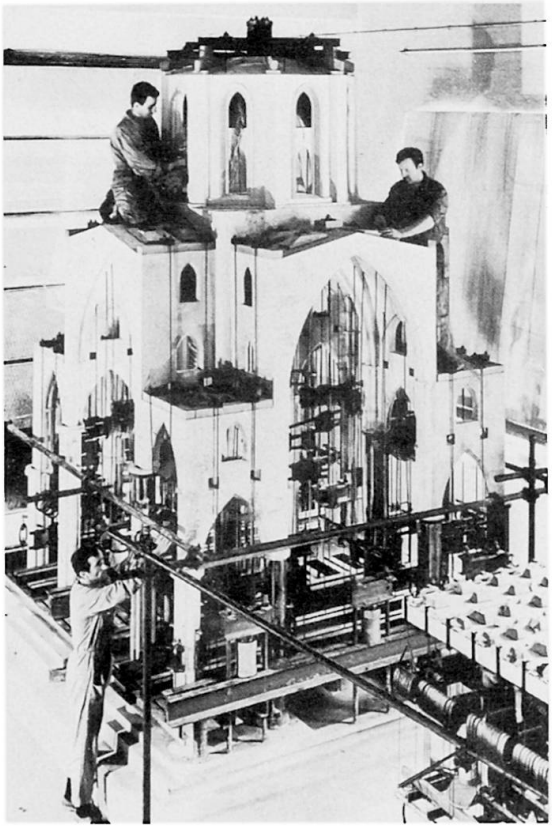

FIG. 8 1:15 scale elastic model of the masonry dome and columns bearing the main spire of the Cathedral in Milan.

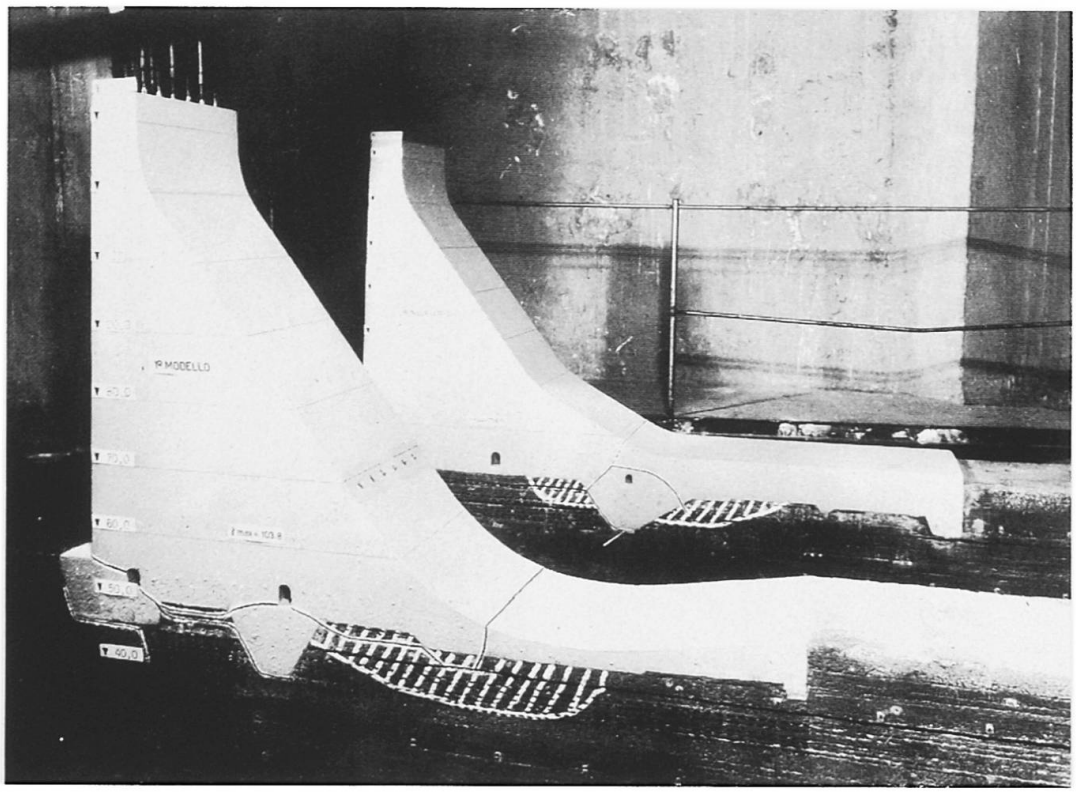

FIG. 9 1:60 scale geomechanical models of a gravity dam in Spain resting on a stratified orthotropic foundation. The tests investigated the effects of the artifices devised to raise the stability of the dam against sliding.

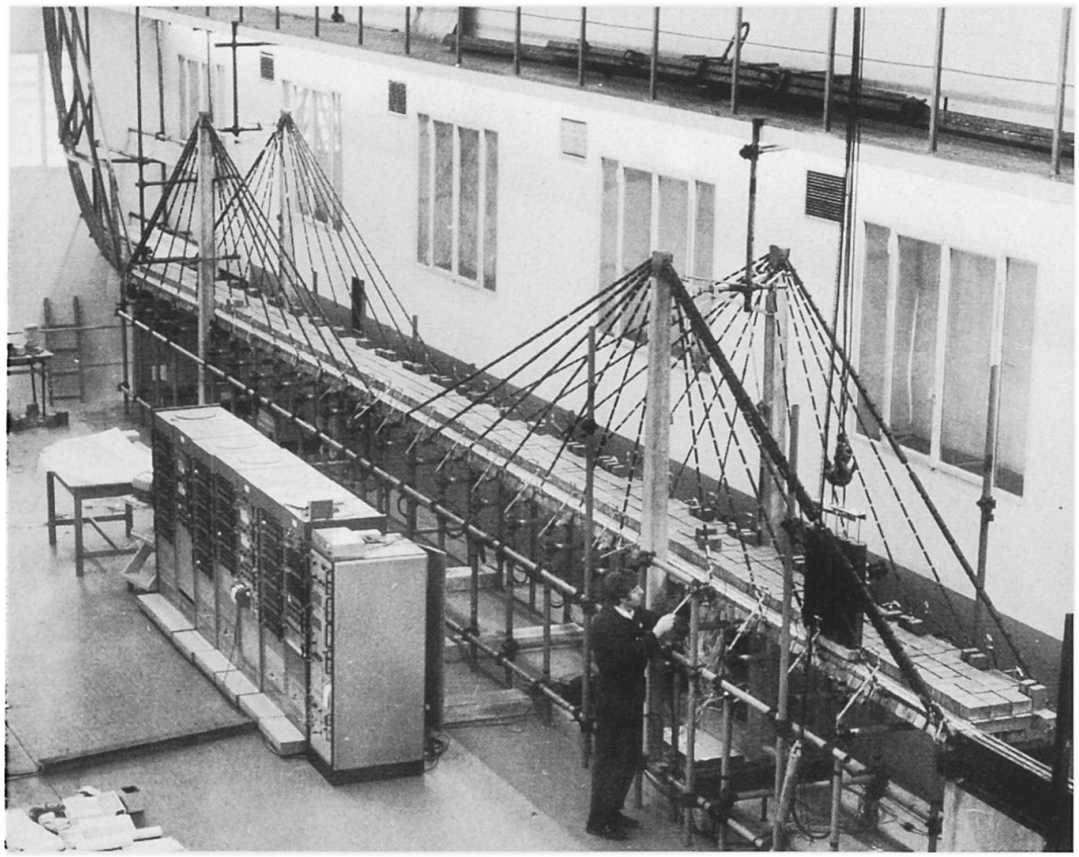

FIG. 10 1:33 scale model of road and railway guyed bridge over the Rio Paranà, Argentina, (complex steel structure). Statical and dynamic tests (running through of a train).

Application of Precast Reinforced Concrete and Steel for Long Span Highway Bridges. Economical Considerations.

Application du béton précontraint et de l'acier aux ponts routiers de grande portée. Considérations économiques.

Spannbeton- und Stahlanwendung für die Autobahnbrücken grösseren Spannweite. Wirtschaftliche Aspekte.

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Generally two materials - steel and prestressed precast reinforced concrete are used for long span highway bridges. Experience obtained in the USSR and other countries during construction and design of these bridges allows to determine an effective area of application for superstrustures of different forms using as the main index - the cost per sq.meter of deck area. This envisages that the index takes into account:

- unit cost of bridge construction work accounting requirements in temporary auxiliary assembly facilities;
- unit cost of main constructions of superstructures of different systems;
- unit cost of main construction of piers corresponding to the chosen system of superstructures.

Thus, the index takes into account all factors such as: cost of material and the fabrication of superstructure and piers and cost of bridge erection.

It is interesting to note that technological factors have considerable impact on final cost of the bridge.

This research indicates that depending on the chosen erection method the cost of auxiliary assembly structures and facilities (piers, falsework, trestles, pontoons, etc.) approximates from 70 to 20 per cent of the cost of superstructures. Consequently

the wrong choice of assembly method can lead to the cost of superstructures erection being equal to the cost of main construction.

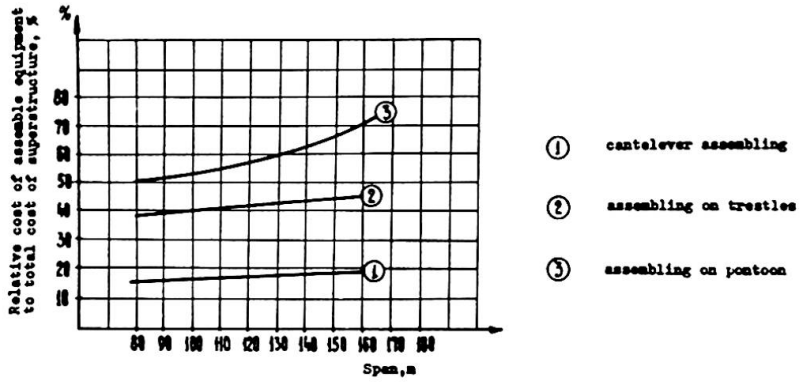


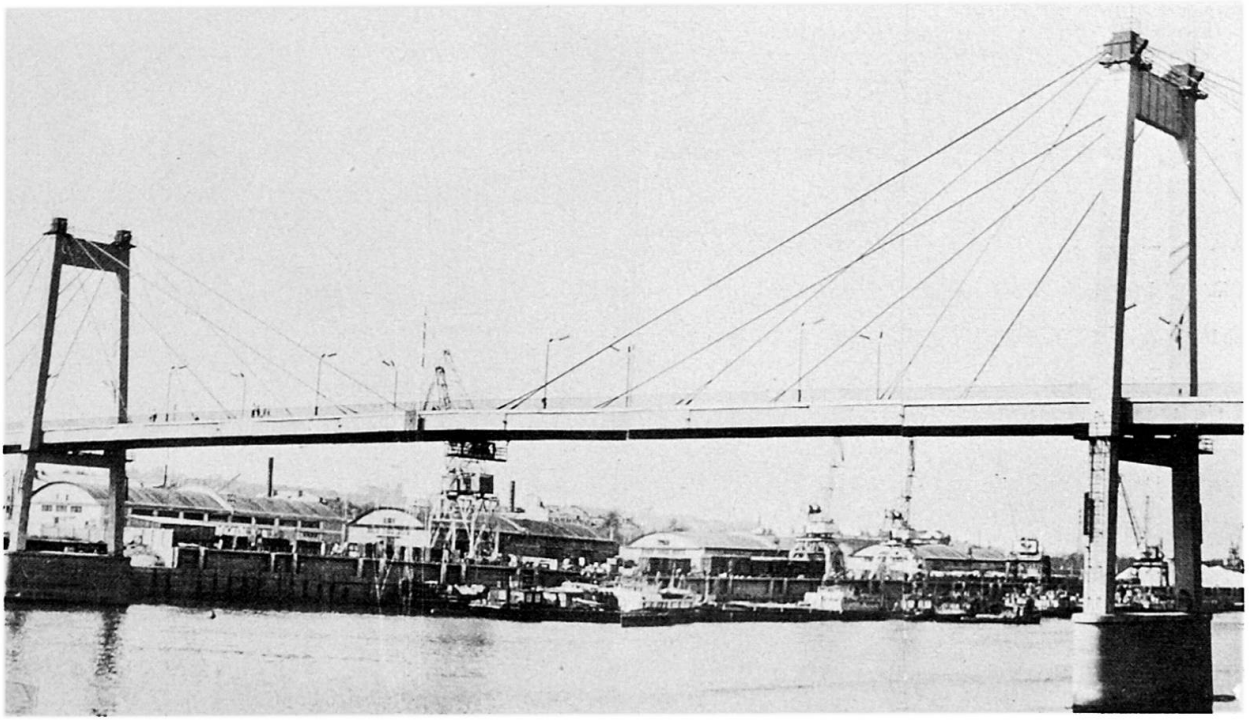
Fig. 1 Relation of assembly equipment and total cost to span of the bridge

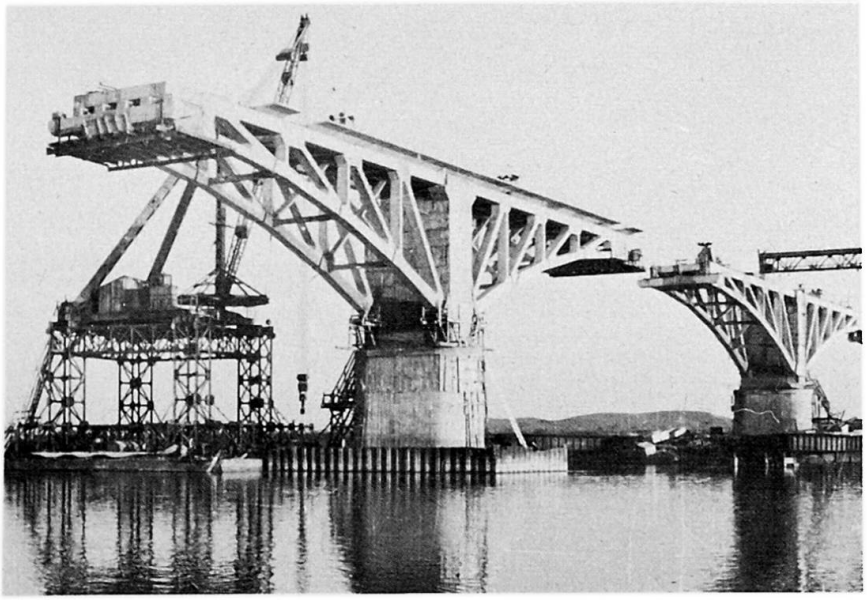
Figure 1 shows graphs of dependence of assembling facilities relative cost upon the cost of main constructions of superstructures of bridges with different spans. Most of the large bridges in the USSR are being erected this way due to the evident technological and economic advantages of cantilever assembly method.

The analysis of the main constructions cost included a review of the most promising superstructures:

I. Reinforced concrete bridges: continuous, rigid-frame cantilever, arch cantilever (Fig.2), arch, cable stayed (Fig.3), rigid-frame suspension (Fig.4).

II. Steel bridges: continuous girder, arch, cable stayed, suspension.





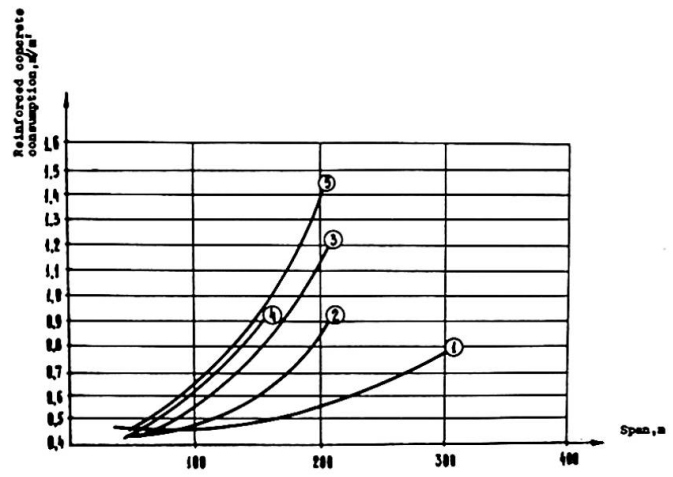


Fig. 5 Relation between reinforced concrete consumption and span of the bridge

The consumption of steel and reinforced concrete per 1 sq.m of superstructures of various systems is shown in Fig.5 and Fig.6. The curves are drawn in accordance with the minimal consumption reached in practice of designing and construction. Then the specific material consumption for the bridge piers of various systems (Fig.7) was analysed, because the account of material consumption only for superstructures without respect for the consumption of material on piers can lead to serious errors while evaluating the chosen structure. The proposed approach to the evaluation of the economic effectiveness of various bridge systems made of steel and reinforced concrete is not expressed by the comparison of the steel and reinforced concrete consumption as a whole,

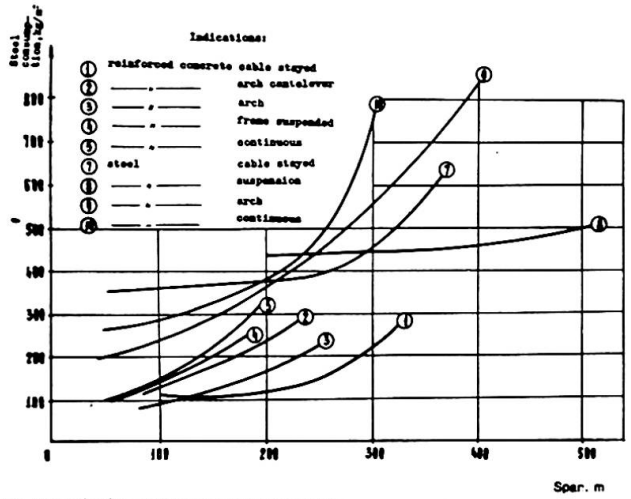


Fig. 6 Relation between steel consumption and span

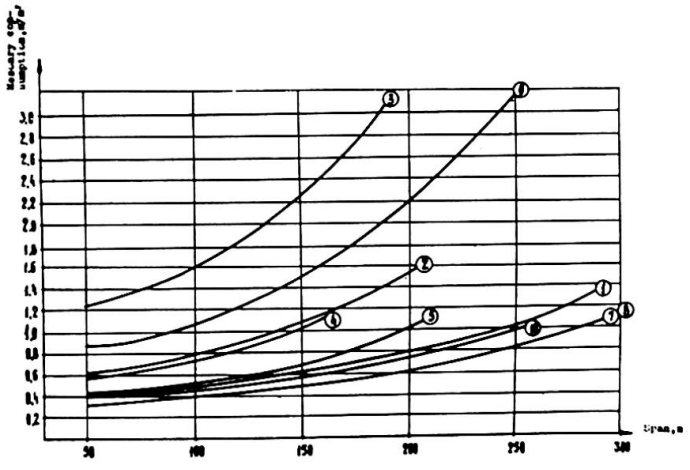


Fig. 7 Relation between masonry consumption and span

but is revealing the area of rational application of each system based on the comparison of certain specific consumption of materials for structural elements depending on spans and the real unit cost of these elements which takes into account the expenses both for manufacturing and erection of structures.

As a result of this analysis a graph was made for the unit cost per sq.m of various steel and reinforced concrete bridges. It includes the costs of construction of superstructures and piers and auxiliary assembly structures and facilities (Fig.8).

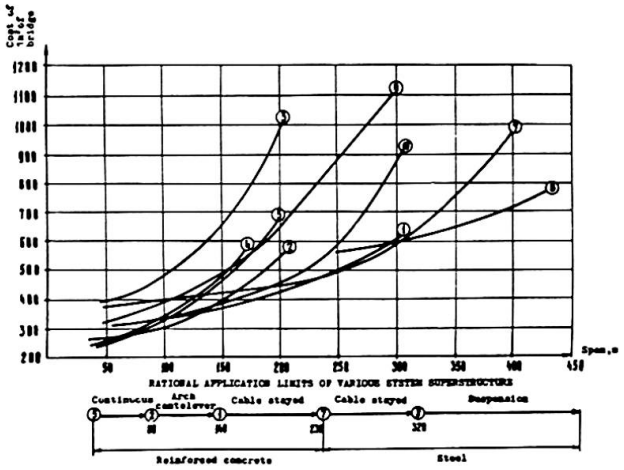


Fig. 8 Relation between cost of 1 m² the bridge and span

It's seen from the graph that at present, under the existing real prices one can speak about the rational application of reinforced concrete bridges of certain most effective systems with spans up to 230 m. These are the following:

- continuous superstructures with box girders erected by the cantilever method with spans from 50 to 100 m made of prestressed concrete;
- arch cantilever structures erected by the cantilever method with spans from 80 to 150 m made of prestressed concrete;
- cable stayed superstructures with the reinforced concrete rigid girder erected by the cantilever method with spans up to 140-150 m.

Then comes the area of the application of various steel bridges-cable stayed and suspension ones.

Naturally the local conditions of construction can considerably influence the definition of the rational application of the above systems. Thus, the availability of works located close to the surface or the construction of single-span bridges over deep canyons will require an additional study of the possibility to use thrust systems, since the volume of materials to be used for piers under these specific conditions is sharply reduced.

For example, when bridges are erected under conditions of city development where the construction of high access is not expedient, the structures of continuous girder or cable stayed bridges of small structural height are preferable.

The proposed method for the evaluation of the economic effectiveness of systems does not exclude real local conditions which are quite different from those mentioned in the present article, but on the contrary, it takes them into due consideration. For the overwhelming majority of bridge superstructure erected under the conditions of plain rivers with soft soils it is reasonable to continue persistent work at the modification and development of the most perspective reinforced concrete systems using them for spans up to 250 m.

SUMMARY The article presents method of choosing the area of rational application of bridges with reinforced concrete and steel span structures of various systems. It considers both the cost of main bridge structures (superstructures, piers, etc.) and the cost of assembly works, accounting necessary temporal structures (falseworks, pontoons, etc.). Cost index per sq.meter of deck area is used as effectiveness criterion.

RESUME L'article présente la méthode de la détermination d'application rationnelle des ponts en béton armé et en acier des systèmes différents. On y tient compte du coût des constructions principales du pont telles que la superstructure et les piles aussi bien que du coût des travaux de montage y compris les ouvrages proviscires (piles provisoires, coffrage, pontons, etc.) Comme le critère d'efficacité on utilise l'index du coût d'un mètre carré du tablier du pont.

ZUSAMMENFASSUNG Im Beitrag wird die Methode der Bestimmung der rationellen Anwendung der Brücken mit Stahlbeton-und Stahl-Spannweitenbauten der verschiedenen Systemen beschrieben. Dabei wurden sowohl die Kosten der führenden Brückenkonstruktionen (Brückenspannungen, Stützen) als auch die Kosten der Montagearbeiten unter Beachtung der notwendigen Behelfsbauten (Baugerüst, Brückenschiff u.a.) berücksichtigt. Als Kriterium der Effektivität wird hier die Wertkennziffer eines Quadratmeters der Brückenfläche benützt.

II

Progrès dans l'optimisation structurale

Fortschritte in der Optimierung von Tragwerken

Progress in Structural Optimization

II a

Concepts et techniques d'optimisation

Grundlagen und Methoden

Optimization Concepts and Techniques in Structural Design

II b

Optimisation des systèmes et des dimensions pour des comportements structuraux linéaires et non-linéaires

Optimierung der Systeme und der Abmessungen bei linearem und nicht-linearem Verhalten des Tragwerkes

System and Geometrical Optimization for Linear and Non-Linear Structural Behaviour

II c

Exemples de calculs d'optimisation à l'aide de l'ordinateur

Beispiele des Computer-Einsatzes bei der Optimierung

Examples of Computer-aided optimal Design of Structures

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Comments by the Author of the Introductory Report

Remarques de l'auteur du rapport introductif

Bemerkungen des Verfassers des Einführungsberichtes

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Optimization Concepts & Techniques in Structural Design

In preparing the Introductory Report for Theme II, 'Progress in Structural Optimization' the three reporters each took a specific aspect of the theme. My objective was to provide an introduction to Structural Optimization; to describe its philosophical goals and to outline in brief and simple terms some of the mathematical techniques which are most frequently used to solve structural optimization problems. It was intended to be a 'beginner's guide' to the topic which would be expanded in more detail by the other Reporters and by the authors of papers in the Preliminary Report.

I will not dwell on an introduction to structural optimization but will assume that you are familiar with it from the Introductory Report. Towards the end of the Report I have remarked that by 1970 most of the simple problems of structural optimization had been solved, only the difficult ones being left. I think this point is demonstrated very well by the six papers in the Preliminary Report under Theme IIa. None of them deals with simple, straightforward problems; they all are concerned with difficult aspects of the topic and they all give a very fair indication of the present-day complexities of structural optimization.

I would like to consider first the papers by Anraku and by Balasubramonian and Iyer since they represent the forefront of technologically difficult problems. Anraku is concerned with designing steel frames to withstand dynamic earthquake loadings. Balasubramonian and Iyer are concerned with random variable variable loadings. It is significant that throughout my Introductory Report I have not mentionedloadings such as these. Much of the research done in structural optimization over the last twenty years has considered only deterministic static loads. Only in the last few years have researchers begun to look at optimum design for dynamic or non-deterministic loads. The reason for this is that the complexity of the problems increases dramatically as one moves away from deterministic static loads and this is an area of work which still requires much research. It is also an essential area for future research. Structural design methods and codes of practice are now moving towards a greater recognition of the probabilistic nature of loadings. Limit state concepts in which safety factors against many different possible occurrences are assessed are also becoming widely accepted. If structural optimization is to retain any relevance for the practising engineer it is essential that it too should be able to handle dynamic loads, probabilistic loads and limit state concepts.

The papers by Anraku and by Balasubramonian and Iyer are therefore welcomed because they are pointing in the right direction for the future of practical structural optimization. However, Balasubramonian and Iyer's paper dealing with structural optimization under random loading effects is entirely theoretical. One of the main reasons why structural optimization methods are not now used more widely in practical design is that there has too often been a large gap between what is correct in theory and what works in practice. This is particularly so when using applied probabilistics. There is a world of difference between defining in theoretical terms the probability of failure of a structure and actually evaluating it accurately for a real-world structure. Nevertheless Balasubramonian and Iyer have made a start in rationalising the effects of random loadings.

Anraku's paper deals with optimum design of frames for earthquake loading - once again a technologically complex form of loading. Some codes of practice incorporate requirements for designing against earthquakes and Anraku is to be complimented on attempting to extend structural optimization into this difficult area of work. As an optimization method Anraku has used sequential linear programming. This method is often used when no other method is available or when the problem is very complicated. Unfortunately it is on these highly nonlinear problems that its performance is worst and it is evident from Anraku's paper that he has experienced difficulties with this method. He comments that an accurate analysis of the dynamic loading is essential if the method is to converge and it appears from his Figure 6 that his optimized design violates some design restrictions by as much as 20%. Both these effects are inherent in the sequential linear programming method. Any linearisation of a highly nonlinear model is bound to be both sensitive to error and inaccurate.

The paper by Brozzetti *et al* is a complete contrast to the preceding papers. It is concerned with a very practical, pragmatic approach to using a commercially available computer package program for designing steel structures. In particular they consider the minimum weight design of practical steel frames so as to satisfy a large number of limit state criteria. The paper highlights the philosophical point that structural optimization is not a mathematical discipline but is, and will always continue to be, an engineering discipline. The objective of structural optimization is to produce the best possible engineering structure. Sometimes precise mathematical methods will allow this to be done mathematically but usually the practical limitations of codes of practice, methods of construction and aesthetics.make a completely mathematical formulation of the design problem impossible. Here the expertise of the engineer is essential. Sometimes those researching new structural optimization methods ignore practical considerations or make dubious assumptions in order to force a practical problem into a mathematically amenable form. While this may be possible for research purposes it is not possible for practical design purposes. Practical structural optimization very often has to be an inexact process relying sometimes upon rigorous mathematics, sometimes upon heuristics and always relying upon engineering experience. Brozzetti *et al* do not describe their optimization technique in detail - it seems to be sequential linear programming but coupled with a lot of engineering knowledge in order to produce real-world structural designs. In their paper they demonstrate that in order to produce really economical designs it is necessary to include the nonlinear interactions of axial forces and bending moments in steel framed structures. Very often these interactions are ignored by researchers when studying these structures since they introduce awkward mathematical nonlinearities.

The remaining three papers all deal with almost classical topics in structural optimization. Structural optimization has always been concerned with two basic questions - one practical, the other more theoretical. The practical question is - 'How can I design the most efficient structure to

perform a specific task?' The more theoretical question is - 'What are the funadmental laws which govern structural efficiency?' It is important to distinguish between these two questions and theoretical work which attempts to answer the second question should not be criticised because it seems irrelevant to practical design. Work in theoretical structural optimization is important and essential because it adds to our fundamental knowledge of structural behaviour. An increased awareness of why some structures are more efficient than others will eventually benefit practical design engineers but the immediate practical relevance of such work may not be apparent.

Nakamura and Nagase consider the optimum rigid-plastic design of multi-storey plane frames for multiple load cases. In my Introductory Report I mention in Section 4.4 that optimum rigid plastic design can be represented as a linear programming problem. Nakamura and Nagase have done this and have then considered some of the more advanced aspects of linear programming theory using duality theory in order to reduce the size of the problem and solve it rapidly. This area of work, optimum rigid-plastic design is much researched and it can truthfully be said that our knowledge of the mechanics of structures in the plastic régime has been greatly advanced by such work. Nakamura and Nagase have made an important contribution to this topic by considering multiple loading cases and their paper is well worth further study. They do not claim to be able to produce an optimum practical design but their method can be used for rapidly producing an efficient and economical initial design which can then be analysed and modified in minor ways to satisfy engineering criteria. I commend their treatment and uses of duality and I believe their work could be developed to form the basis of really efficient design programs for practical structural design.

The two final papers, one by myself and one by Lipp and Thierauf both deal with the same classical problem. How can one design truss-type structures for minimum weight in the presence of restrictions upon member stresses, nodal displacements, member size limits? The optimum design of trusses has always been a subject of much research for several reasons. First of all trusses are practical engineering structures and so it is a relevant area of work. Secondly, the problem is a nonlinear one of a most interesting mathematical form and thirdly the methods which can be used to design trusses can also, with minimal modifications, be used to optimally design certain classes of more complex finite element plate structures. Perhaps the major difficulty which any optimum truss design method has to face is that of problem size. For each truss member there is usually one variable (the cross-sectional area) for which an optimal value is to be found. Trusses of several hundred members are not uncommon so for these structures the optimum design problem expressed mathematically is nonlinear, has several hundred variables and even more constraints.

A straightforward numerical search for an optimum of such a large problem is not possible as it is wasteful of time and computer resources. Recently engineers have looked more deeply at this problem and have found that by examining the theory of optimality more carefully new, more rapid design methods for trusses can be developed. My own paper explores this topic further and describes how duality principles can be used to develop new design methods. The paper by Lipp and Thierauf is concerned with the same approach - indeed the mathematics of the two papers is remarkably similar. I do not have time in this summary to talk about the differences and similarities in these papers in detail but I would like to add a final comment. In my Introductory Report I mention that duality might prove to be a mathematical concept of great value to those interested in the optimum design of large structures. My own paper reflects this of course but it should be noted that the Lipp-Thierauf paper is also concerned with duality via the Lagrange multiplier technique thus strengthening my earlier opinion.

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Comments by the Author of the Introductory Report

Remarques de l'auteur du rapport introductif

Bemerkungen des Verfassers des Einführungsberichtes

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System and Geometrical Optimization for Linear and Non-Linear Structural Behaviour

Please accept by apologies for being unable to attend and report personally to this meeting but circumstances beyond my control have led to my absence. I offer my best wishes for the success of this session and I thank in advance Dr. Templeman for substituting on my behalf.

Many discussions at earlier IABSE sessions have considered optimization. Professor Courbon defined optimization as designing and constructing a structure at the lowest cost with the object of fulfilling a well-defined purpose. Cost consideration must be given to safety, service life, maintenance and future adaptability. Within this broad context, the speciality of structural optimization arose to provide specific design purposes and methods which will aid in reaching an optimum structure. Thus, in the same way that matrix methods or finite elements aid in structural analysis, techniques of structural optimization have been developed to improve design procedures. Its applicability depends as much on reducing the ultimate cost of the structure as on savings in time and cost for the design engineer.

Historically, optimization has used simple design rules to check optimum designs. Gradually, more sophisticated mathematical methods applied with computer programs arose to systematically search and locate optimum structures.

A description of formal optimization methods taken from fields of mathematical programming and Operations Research has been presented by Dr. Templeman in his survey paper published in the Introductory Report. Such methods have found widespread application in the design of structural elements which are described by a number of design variables and constraints determined by codes of practice.

Figure 1 of my Introductory Report shows examples of such element designs. There is an example of a welded box girder for which I have had occasion to design large numbers for crane structures. Another example, is the welded plate girder which we designed based on the rather complex provisions for unbraced members in the AISC specifications. Also shown is a prestressed concrete beam with eleven design variables. The element design is controlled by contraints on loading and prestress force and deformations. Other element designs reported include welded columns, stiffened ship plates, shear walls, prestressed plates and reinforced concrete beams.

Element optimization has led to a number of computer programs whose function is to efficiently design a variety of elements and perform the tedious calculations required by the designer in trying to proportion such elements. The programs have usually been based on penalty or geometric

programming methods of optimization. Professors Ohkubo and Okumura in their Preliminary Report paper have derived the optimum design of elements such as bridge girders and truss members using the method of sequential linear programming. This was then adapted by them to a branch and bound procedure for solving discrete variables such as steel type and flange thickness. A different approach to the optimization of element, in this case concrete bridges, is presented by Ulizkij and Jegoruschkin. It uses influence factors for predicting the behaviour of the bridge and therefore simplifies subsequent optimisation.

A combination of elements as in a total structural framework requires a different approach to optimization. Any changes in the design on the path to the optimum may subsequently require complete reanalysis of the structure to determine new stresses and deflections. In Figure 2 of my Introductory Report, a grillage is shown in which redistribution of forces occurs following each design change. The optimum design procedure for this case was reported by Moses and Onoda. Other examples of system optimization are statically indeterminate trusses and frames.

A system optimization, to be efficient, requires techniques such as the sequential linear programming shown by Ohkubo and Okumura. It is important that the number of cycles of reanalysis does not become large leading to excessive demands for computer time.

Inclusion of gross geometrical variables of the structures represents an important improvement in the class of problems for which optimization may be applied. Figure 4 of my Introductory Report shows a transmission tower in which the tower shape and location of nodes is permitted to change leading to significant reductions in structural weight. The left figure is the original design while the right is an optimized case. The optimization takes place automatically with a program using methods of minimisation working with respect to the geometric or shape design variables.

Another example of geometric optimization is the arch dam reported by Vitiello and shown in Fig. 5 of my Introductory Report. The mesh shown is the finite element analysis while X_1 - X_4 are the geometric design variables. Such applications show that major improvements in structural efficiency can often come from variations in geometric design variables. This is investigated for arches and suspension bridges by Professor Hirai and Yoshimura in their Preliminary Report.

Form and type of structure represent a high level of optimization for which programs have only recently been attempted. Figure 6 from my Introductory Report paper shows a schematic diagram for optimizing the cost of single storey factory buildings. The variables include structural layout such as bay spacing and also the type of joists, girders, columns and foundation including material type and detailed design variables. The design methods, automatically performed by the computer, can lead to important structural savings and can be updated following changes in individual construction and material costs.

Bomhard in the Preliminary Report shows a comparison of structural form with an illustration of beams, arches and suspensions to cope with long-span structures. Suruga and Maeda have developed a very interesting concept of a decision matrix to compare structural forms for their application to floor systems of long-span bridges. Each type of floor system such as composite girder or orthotropic deck is rated according to cost, construction and performance before a final decision can be made. This leads to a multi-objective criteria for optimization which may have important applications to other examples such as comparing economy with safety. The inclusion of safety directly in the optimization methodology is covered by Tegze and Lenkei with an example of collapse analysis of statically indeterminate plane structures.

The inter-relationship of safety and economy of structure has been recognised by many authors, but more effort is required to bring these factors into both the code specifications and the programs for optimum design.

Comments by the General Reporter

Remarques du rapporteur général

Bemerkungen des Generalberichterstatters

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Examples of Computer-Aided Optimal Design of Structures – General Report

The main subject of Theme II is on "Progress in Structural Optimization", which was originally proposed by the Japanese National Group of IABSE, because it was intended to stimulate and encourage Japanese engineers to apply the concept and method of optimization to problems of structural design, since the structural optimization has very recently been introduced into Japan. Along this intention we are very thankful for the three excellent Introductory Reports.

Dr. Gellatly and Mr. Dupree presented a very excellent paper as an introductory report on applied structural optimization in terms of examples of computer-aided optimal design of structures. They covered two different approaches to the optimum design of complex structural systems, emphasizing the practical aspects of design problems intended for producing a useful tool for designers.

The first approach, "Optimality Criteria Approach" will be accepted by designers because of its simplicity and effectiveness. The approach to the weight minimization of fixed-geometry structures with constraints based on the use of optimality criteria, appears to offer considerable advantage over mathematical-programming based methods. At comparative studies, the present method seems to reach a similar or better design in considerably fewer iterations than most numerical search methods with the reduction of computational costs.

They presented five examples, and also Dr. Gellatly discussed this approach at his other paper[1)] at an example of "Twenty-five-bar Transmission Tower" in which, using the current program, convergence was obtained in seven iterations

to get its minimum weight, although, using a numerical search method, over one hundred analyses might have been required.

These results are very encouraging us, because they indicate that some, if not all, of the difficulties encountered in large-scale optimization problems for the very large number of variables in finite element representation of real structures, can be eliminated through this type of approach. However, certain problems may still remain to be unsolved, particularly with regard to convergence characteristics.

The second approach is labeled Sieve-Search Procedure developed at Bell Aerospace Company, the guiding philosophy of which is that an optimum system is an optimum arrangement of pre-optimized components. The results obtained from the design studies on high-speed vessels and a design study on a complete bridge structure have indicated that, firstly, the method will permit the full variation of construction method, materials and configuration as well as component sizing, and secondly, this method is also an efficient cost-effective approach to automated optimum design.

Dr. Gellatly and Mr. Dupree suggested finally that the ideal solution for optimization problems would possibly appear to be a combination of the two approaches, in which the sieve-serach defines configuration and non-continuous variables and the optimality criteria method will be used for refinement of the design, expecting a considerable potential for overall system optimization at various design problems.

We have been expecting a number of papers to be presented at the Preliminary Report under the stimulus and for the discussions of the Introductory Reports. For the Sub-Theme IIc, the following five papers have been accepted:

1. The paper presented by Mr. Gurujee

The paper shouldhavebeen discussed at Theme IIa. He proposed a general optimization algorithm for a structure. A structural optimization problem can be generally solved as a sequence of analysis-programming cycles by the mathematical programming. In the optimization process which the author proposed in the form of a chart shown in Fig.1 at the Preliminary Report, p.179, the relation between the changes in the behavior variables due to a specified change in each of the design variables, is found and stored in the form of "Sensitivity Matrix". Then, the programming problem can be solved by using the penalty function method. In this paper, however, he did not show any specific examples

to which his proposed method was applied.

2. The paper presented by Prof. Yamada and Mr. Furukawa

They treated the optimal design of a system of tower and pier of a suspension bridge, on the elastic foundation subjected to earthquake ground motion. They showed an example how to combine mathematical programming and dynamic structural analysis through response spectrum for a dynamic loading problem, referring to Figs. 1 and 2 at the Preliminary Report, p.184. To simplify very complicated real dynamic behavior of the system, two design variables were selected: longitudinal width of the pier and stiffness of the tower. A generalized cost was selected as the objective function, and requirements for stress of the tower and displacement of the pier at its top, and buckling of the tower, overturning of the pier, and physical limits, were constraints.

Since the problem is non-linear and undifferential, the Sequential Unconstrained Minimization Technique by Powell's direct search method was applied to optimization, probably because the method is more reliable in terms of guaranteed convergence if the first derivatives or no derivatives are available. At a numerical example, the authors found out that the generalized cost is greatly affected by the modulud of elasticity of the foundation. This problem is overall system optimization of a simple tower-and -pier system. Shape and geometry optimization and combination with detailed element optimization will be a future problem.

3. The paper presented by Prof. Konishi and Prof. Maeda

The paper on "Total Cost Optimum Design of I-Section Girders for Bridge Construction" treated examples of detailed design optimization of main elements of girder bridges. Generally, at the problem of bridges, cost optimization is selected as the objective function, but the cost used to be defined material cost only or material plus overall fabrication cost. At the present paper, the objective function consists of material and fabrication costs, which cover costs of full-scale drawing, machining, shop welding, shop assembly and shop painting base on actual detailed informations obtained at fabricating shops in Japan.

A computer-aided optimum design of girders by the method of "Sequential Linear Programming" was illustrated at I-shaped, deck-type, welded plate girders with five different span lengths, and sixteen design variables including material selection (See Fig.2 at the Preliminary Report, p.192). The influence

of material and size selections on the total cost was discussed in detail, to help designers carry out a detailed element design efficiently from the point of optimization, taking into consideration not only material cost, but also shop fabrication cost.

For a specific or individual bridge, it would be required to study on an overall optimization design including transportation and erection costs for a system of main girders, laterals and decks.

4. The paper presented by Professor Schindler

He proposed an optimization method to combine design-oriented approach and computer-oriented approach, in which a designer can search for a range of approximation near an optimum value with a design program, within the capacity of a computer, not spending so much money and time for computer calculation.

He illustrated his method at the optimum design of a railway truss bridge shown in Abb.1 at the Preliminary Report, p.196, taking into account three kinds of deck system , two kinds of steel, two kinds of bridge class, five kinds of span length. The objective function was total steel weight, and the design variables were span length, number of panels, height of the truss, and width of chord members. For various truss heights, steel weights were calculated by a computer with parameters of span length and number of panels. By comparison of each steel weight, the minimum weight was found out for a certain value of span length and of number of panels.

This approach is not straightforward, but rather comparative or selective. Sometimes depending on a problem, this approach may save the time and money for a computer more than mathematical programming methods. This kind of approach could be examined in contrast with a study presented by Prof. Ohkubo at Theme IIb [2] who proposed a sub-optimizing method for trusses.

5. The paper presented by Messrs. Tanaka, Kamemura and Maruyasu

They introduced the total computer-aided design system for girder bridges. which has recently been developed at Nippon Kokan Company, Japan. Automated computer techniques for design have advanced so that various types of detailed element design and selection among alternatives for minimum cost can be carried out. In this sense, the proposed computer system is a well advanced method for automated design of a girder type bridge in its element and overall system.

As the authors pointed out, such a computer program could be used for lowering cost, increasing standardization of elements and also evaluating the effects of changing constraints on weight, cost and behavior. The authors discussed conceptually the interaction between optimum design and automated design, but they did not show concretely with an illustration how to incorporate optimization into the automated design program.

The proposed computer system should be examined in contrast with the flow chart of Sieve-Search Optimization for bridge design proposed by the Introductory Reporters, Dr. Gellatly and Mr. Dupree [3].

As a concluding remark, at the Prepared Discussion nore demonstrations of structural optimization are welcome in terms of examples to encourage designers to utilize optimization techniques at their routine office practice, and also to discuss what kinds of problems have been encountered at practical designs.

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2) S. Ohkubo & T. Okumura, "Structural System Optimization Based on Sub-optimizing Method of Member Elements", Prel. Rept. 10th Congress, IABSE,1976.
3) R.A. Gellatly & D.M. Dupree,"Examples of Computer-Aided Optimal Design of Structures", Introductory Rept., 10th Congress of IABSE, 1975.

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Optimum Design of Cable-Stayed Bridges using an Optimality Parameter

Calcul de ponts haubannés à l'aide d'un paramètre d'optimisation

Die Berechnung von Schrägseilbrücken mit einem Optimierungsparameter

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1. INTRODUCTION

The optimization discussed in this paper is applied for the design of the overall super-structure of cable-stayed bridges. Then the hierarchy of this study is belonged to category 3 described in the introductory report of the 10th congress by Templeman[1]. The optimization method developed here is a kind of the optimality criterion method discussed by Templeman, Gellatly and Dupree[1][2].

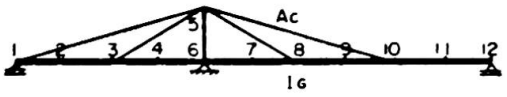
Up to the present many nonlinear programming techniques have been developed and applied for the optimum design of bridge super-structures, but successful applications are very few. Because the fully stressed design for a common type of super-structure such as girder bridge is a convenient design method and gives the satisfactory economical result. Therefore from the practical point of view, the optimum design without considering the price of sub-structure may be important for only some specific type of bridges such as cable-stayed bridges, suspension bridges.

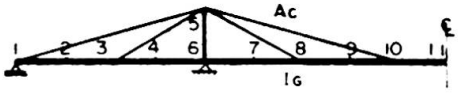
In this study, an optimality condition parameter is obtained by a mean of the numerical calculation and the parameter is used to determine the economically proportional sizes of the cable and girder.

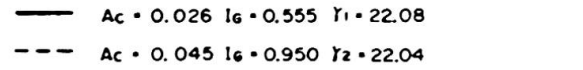
2. PRECONDITIONS FOR THE NUMERICAL PROCEDURE

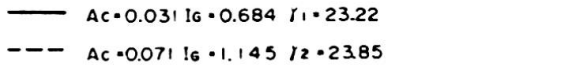
To determine an optimality parameter by numerical process, the following preconditions must be given.

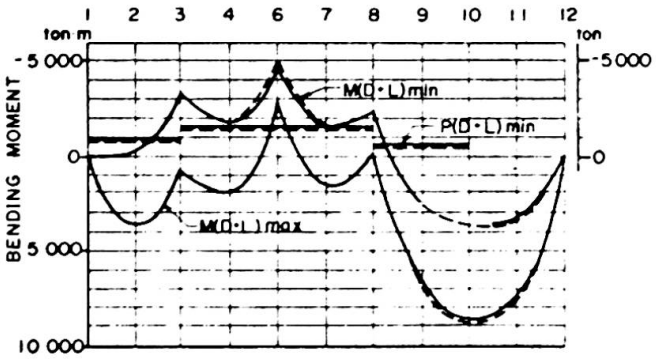
(1) Utilization of the structural nature of cable-stayed bridges is very important to find out the optimality condition. Fig. 1, 2 show the static behavior of cable-stayed bridges due to dead loads and live loads. These examples show that the each rigidity value of cable and girder is not main factor of changing the section force distribution. It is obvious that the main influence to the girder section force is a rigidity ratio, γ = EG·IG/EC·AC. where, EG is the modulus of elasticity of girder, IG is the moment of inertia of girder, EC is the modulus of elasticity of cable, AC is the cable area.











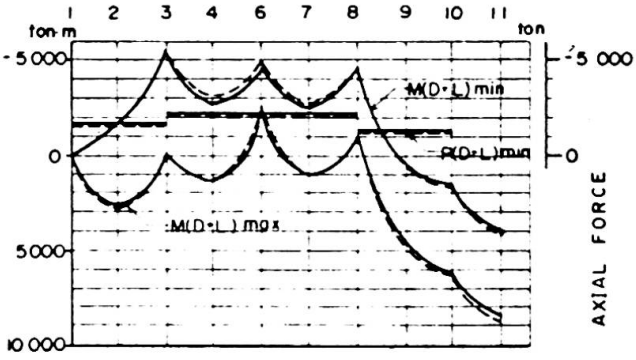


Fig. 1 2 Span Bridge

Fig. 2 3 Span Bridge

(2) The price ratio of materials including the cost of fabrication and erection is assumed as follow.

structural steel : high tensile steel : cable = 1 : 1.15 : 2.0

(3) Fig. 3 is the analyzing structural system which rigidity ratio is assumed as:

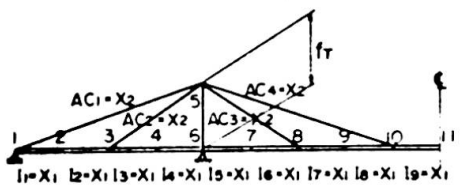
$$\gamma_{try} = EGX_1/ECX_2$$

X_1: moment of inertia of girder
X_2: cable area

Fig. 4 is the actual redesign structure which rigidity ratio is expressed by:

$$\gamma_{real} = \frac{EG/NG \cdot \sum_{n=1}^{NG} IGn}{EC/NC \cdot \sum_{n=1}^{NG} ACn}$$

IGn: moment of inertia of girder
ACn: cable area
NC, NG: number of cable and girder



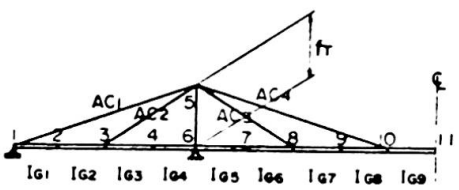


Fig. 3 Assuming Member System

Fig. 4 Actual Member System

The approximate design process must be carried out by keeping the following ctiteria.

$$0.9 < \gamma_{try}/\gamma_{real} < 1.1$$

(4) The section and material compositions of stiffening girder are illustrated in Fig. 5, 6. The price of girder member is determined by the element design based on the fully stressed design.

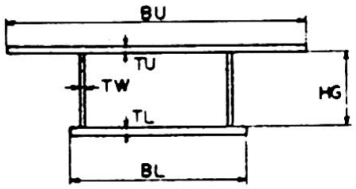


Fig. 5 Girder Section

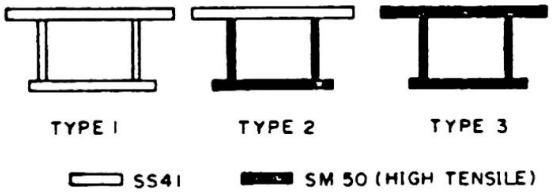


Fig. 6 Material Composition of Girder

3. DETERMINATION OF AN OPTIMALITY PARAMETER

An optimality parameter is determined after carried out the next 2 step procedure.

(1) Characteristic Parameter (Step 1)

The basic structure to be effectively prestressed is determined by the grid search procedure, because two design variables are employed for the global system optimization. The cost evaluation is made by the following equation.

$$Z(X_1,X_2) = \sum_{m=1}^{NG} \text{price } G(X_1,X_2) + \sum_{n=1}^{NC} \text{price } C(X_1,X_2)$$

X_1: moment of inertia of stiffening girder
X_2: cable area
price G: price evaluation of girder depend on X_1, X_2
price C: price evaluation of cable depend on X_1, X_2

The characteristic parameter at the grid point is expressed as:

$$KE = EG \cdot IG / EC \cdot AC \cdot HG^2$$

IG: moment of inertia of stiffening girder
AC: cable area
HG: web depth

(2) Determination of an Optimality Parameter (Step 2)

Prestressing forces (external loads) are introduced into the cable of basic structural system determined by above procedure. In this step, prestressing forces are design variables (Fig. 7). In case of two design variables, an optimality parameter minimizing the total cost is also selected among the grid points number of characteristic parameters, and it is expressed by the following nondimensional parameter.

$$KOPT = EG \cdot IGOPT / EC \cdot ACOPT \cdot HGOPT^2$$

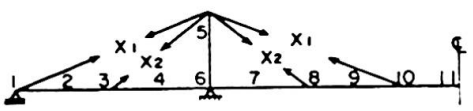


Fig. 7 Prestressing System

4. NUMERICAL MODELS AND RESULTS

Numerical calculation is carried out for nine cases of analyzing models. Structural models of 2-span and 3-span bridges are illustrated in Fig. 8, 9.

The differences of analyzing models are indicated belows.

2-span Bridge

Case	Tower Height	Web Thick	Steel Weight	Other
1	30 m	10 mm	3.3 t/m	
2	30 m	14 mm	3.3 t/m	
3	30 m	10 mm	3.3 t/m	Knie Type
4	30 m	10 mm	5.3 t/m	
5	35 m	10 mm	3.3 t/m	
6	40 m	10 mm	3.3 t/m	

3-span Bridge

Case	Tower Height	Web Thick	KE
7	30 m	10 mm	2.58
8	30 m	14 mm	2.64
9	30 m	14 mm	2.96

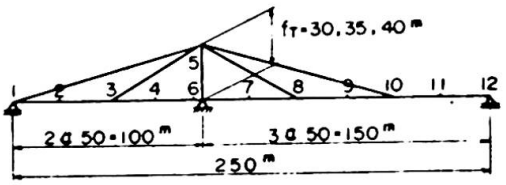


Fig. 8 2 Span Model

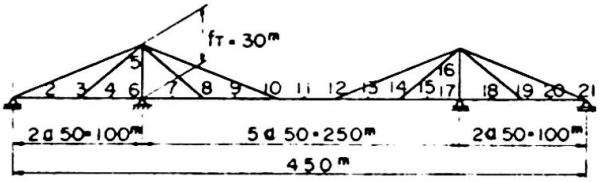


Fig. 9 3 Span Model

Table 1 Numerical Result of Parameters

	CASE	DESIGN STEP	KE PARAMETER	KOPT	PARAMETER
2 SPAN	1	A FULLY STRESS	4.0 ~ 5.0		2.48
		B BASIC STRUCTURE	0.92 ~ 7.56		
		C PS ARRANGEMENT	2.47 ~ 2.89	2.55	
	2	B BASIC STRUCTURE	0.96 ~ 7.88		
		C PS ARRANGEMENT	2.54 ~ 2.96	2.49	
	3	B BASIC STRUCTURE	0.92 ~ 7.73		
		C PS ARRANGEMENT	2.43 ~ 2.90	2.43	
	4	B BASIC STRUCTURE	0.92 ~ 7.03		
		C PS ARRANGEMENT	2.53 ~ 2.89	2.55	
	5	B BASIC STRUCTURE	0.92 ~ 7.05		
		C PS ARRANGEMENT	2.41 ~ 2.89	2.45	
	6	B BASIC STRUCTURE	0.92 ~ 7.50		
		C PS ARRANGEMENT	2.41 ~ 2.89	2.41	
3 SPAN	7	A FULLY STRESS	3.22 ~ 4.73		2.46
		B BASIC STRUCTURE	0.92 ~ 5.67		
		C PS ARRANGEMENT	2.14 ~ 2.58	2.58	
	8	B BASIC STRUCTURE	0.96 ~ 5.81		
		C PS ARRANGEMENT	2.25 ~ 2.65	2.27	
	9	B BASIC STRUCTURE	0.96 ~ 5.81		
		C PS ARRANGEMENT	2.53 ~ 2.96	2.53	
TOYOSATO BR.		SPAN 80.5 · 216 · 80.5 (PWS)			2.87
SUEHIRO BR.		SPAN 109 · 250 · 109 (PWS)			2.89
ONOMICHI BR.		SPAN 85 · 215 · 85 (LOCKED-COIL)			2.39

Numerical results of characteristic parameter and optimality parameter are listed in Table 1. The KE parameter of the cable-girder system determined by the fully stressed design is in the small range (3.∿5.). On the other hand the KE parameter determined by the approximate design process developed in this study is in the fairly large range (1.∿8.). Prestressing forces are introduced into the suitable basic structure which cable components are not fully stressed. From the results of case 7 ∿ case 9, it is obvious that KE value reduces about 15 percents by introducing the prestressing forces.

5. AN EXAMPLE OF THE DESIGN USING AN OPTIMALITY PARAMETER

The main difference of this method from the usual design method is the use of the parameter KOPT = 2.5 obtained by numerical calculation as shown in Fig. 10.

dead load ..... 10.0 t/m
line load ..... 50.0 t
uniform load .... 3.5 t
impact 0.2
optimality
parameter..... KOPT=2.5
assuming rigidity
girder ..... IG=1.0 m^4
cable ..... AC=0.046

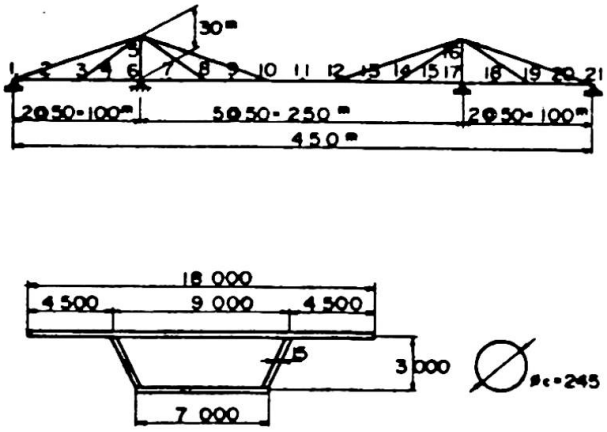


Fig. 10.
Design Conditions
and Basic Dimensions

The optimum bending moment arranged by prestressing forces and the moment inertia of girder members are also illustrated in Fig. 11.

By using the optimality parameter, structural designer can get the reasonable sections of girder and prestressing forces of cable-stayed bridges by one time trial. The assuming rigidity of analyzing system determined by using an optimality parameter is very close to the real rigidity of the final structure.

6. CONCLUSION

Unexperienced structural engineer may feels some difficulties to design the economical cable-stayed bridge. Because allowable stress guarantee the safety of structures, but it does not always guarantee the economical condition. The prices and strength of the cable and steel girder are extremely different. Furthermore the arrangement of the bending stress of the stiffening girder causes the more complicated problem. Therefore the economical criterion for cable-stayed bridges may be important as same as the factor of safety.

Finally the conclusions of the basic study for the optimality criterion method are outlined by the next statements.

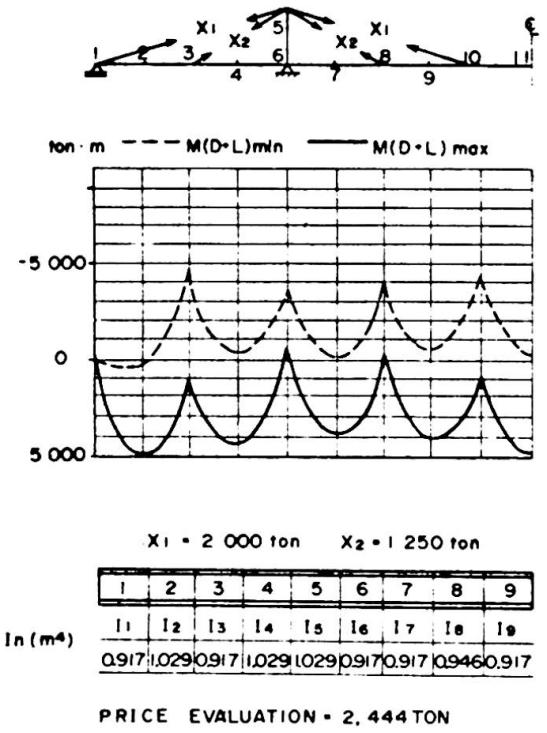


Fig. 11. Optimum Design

(1) The optimality parameter value of the radial type of the cable-stayed bridge based on the price ratio (structural steel : cable = 1 : 2) exists in the range of 2.0 ∿ 3.0.

(2) The moment of inertia of stiffening girder can be considered as uniform along the girder axis by effectively prestressed.

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SUMMARY

A convenient optimization method using an optimality parameter has been discussed. The optimality parameter is based on the structural nature and the approximate design procedure. This parameter is used to determine the optimum rigidity ratio of the cable-girder system after introducing the prestressing forces. The optimum design using an optimality parameter will be easily accepted by structural engineers as the economical criterion.

RESUME

Une méthode pratique d'optimisation à l'aide d'un paramètre d'optimisation est présentée. Le paramètre d'optimisation est obtenu à partir du caractère structural et de la méthode approximative de calcul. Ce paramètre est employé pour déterminer la rigidité la plus favorable du câble et de la poutre après avoir introduit les forces de précontrainte. Le calcul à l'aide d'un paramètre d'optimisation sera accepté par les ingénieurs comme un critère économique.

ZUSAMMENFASSUNG

Eine anwendbare Optimierungsmethode mit einem Optimierungsparameter wird dargestellt. Der Optimierungsparameter wird aufgrund des Tragwerkssystems und der Näherungsberechnungsmethode bestimmt. Mit diesem Parameter werden die günstigsten Seil- und Trägersteifigkeiten unter Berücksichtigung der Vorspannkräfte bestimmt. Die Berechnung mit einem Optimierungsparameter wird von den Ingenieuren zur Steigerung der Wirtschaftlichkeit angenommen werden.

Preponderance of Idealization in Structural Optimization

Prépondérance de l'idéalisation dans les problèmes d'optimisation structurale

Die überragende Bedeutung der Idealisierungen bei der Optimierung von Tragwerken

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The optimal design of a structure may be divided in two steps.

In the first one - the *idealization* - the structural problem is put in following mathematical formulation :

"Find $\vec{X}$ such that :

$$f_k(\vec{X}) < 0 \quad \text{for} \quad k = 1,2,\ldots m \; ;$$
$$h_j(\vec{X}) = 0 \quad \text{for} \quad j = 1,2,\ldots l \; ; \qquad (1)$$

and :

$$F(\vec{X}) = \text{minimum (maximum)} \text{ "}$$

where $\vec{X}$ is a vector which contains the design variables,

f and h are the constraints of the problem,

and F is the objective function to optimize.

The second step - the *solution process* implies (a) the choice of the solving procedure and (b) the search of the solution of the problem formulated as in (1).

In the opinion of the authors, a good idealization is the basic condition for obtaining a good value of the solution, while a more or less refined mathematical treatment of it plays a rather secondary role |1| .

In many papers of the literature, emphasis is too often brought on the choice of the solution procedure rather than on that of a heuristic which does not modify in anyway the sense of the actual problem.

So long as the structural problem is small - about ten variables and constraints - many methods are available in the literature. However, various numerical experiments have shown that the choice of a method depends on the problem to be solved, for most of the algorithms cannot be used economically in all cases |2|. As a consequence, conclusions concerning the use range and the efficiency of an algorithm for a structural problem can rarely be extended to another one.

If emphasis is almost brought on the idealization, the designer may be sure of obtaining a realistic solution of the problem and, in addition, important simplifications in the mathematical treatment of the second step become possible. Indeed, on one way, a judicious choice of variables or an ingenious variable transformation often enable to present the complex problem in a more simple form, and, on another way, by means of a previous evaluation of the several variables, the designer can establish a hierarchy of the variables and divide the complex problem into smaller ones, which are then more easier to solve quickly.

For example, in |3|, MYLANDER demonstrates that a rather simple variable transformation changes a mathematical non-linear and non-convex problem into a linear programming system. It is worthwhile to recall the following basic non-linear problem which is considered as a very difficult one. The objective function is :

$$f(x) = b_o + a_{o1}\, x_1 + \left(\sum_{j=2}^{5} a_{oj}\, x_j\right) x_1 \rightarrow \min$$

subject to constraints :

$$0 \leqslant a_{i1}\, x_1 + \left(\sum_{j=2}^{5} a_{ij}\, x_j\right) x_1 \leqslant b_i \quad i = 1,2,3 \qquad (2)$$

$$x_1 \geqslant 0 \; ; \; 1.2 \leqslant x_2 \leqslant 2.4 \; ; \; 20.0 \leqslant x_3 \leqslant 60$$

$$9.0 \leqslant x_4 \leqslant 9.3 \; ; \; 6.5 \leqslant x_5 \leqslant 7.0.$$

where the values of the constants are :

a_{01}	= - 8,720,288.795	a_{21}	= - 155,011.1055
a_{02}	= - 150,512.524	a_{22}	= 4,360.5334
a_{03}	= - 156.695	a_{23}	= 12.9492
a_{04}	= - 476,470.319	a_{24}	= 10,236.8839
a_{05}	= - 729,482.825	a_{25}	= 13,176.7859
a_{11}	= - 145,421.4004	a_{31}	= - 326,669.5059
a_{12}	= 2,931.1506	a_{32}	= 7,390.6840
a_{13}	= - 40.4279	a_{33}	= - 27.8987
a_{14}	= 5,106.1920	a_{34}	= 16,643.0759
a_{15}	= 15,711.3600	a_{35}	= 30,988.1459
b_o	= - 24,345.0	b_2	= 294,000.0
b_1	= 294,000.0	b_3	= 277,200.0

By putting, according to MYLANDER

$$y_i = x_1 \cdot x_i \qquad i = 2, 3, 4, 5$$

and (4)

$$y_1 = x_1$$

above non-linear problem takes following linear formulation :

$$g(y) = b_o + \sum_{j=1}^{5} a_{oj}\, y_j \rightarrow \min$$

$$0 \leqslant \sum_{j=1}^{5} a_{ij}\, y_j \leqslant b_i \quad i = 1, 2, 3 \qquad (5)$$

$$y_i \geqslant 0 \qquad i = 1, 2, \ldots, 5$$

$$y_2 - 1.2\, y_1 \geq 0 \; ; \quad 2.4\, y_1 - y_2 \geq 0$$

$$y_3 - 20.0\, y_1 \geq 0 \; ; \quad 60.0\, y_1 - y_3 \geq 0$$

$$y_4 - 9.0\, y_1 \geq 0 \; ; \quad 9.3\, y_1 - y_4 \geq 0$$

$$y_5 - 6.5\, y_1 \geq 0 \; ; \quad 7.0\, y_1 - y_5 \geq 0.$$

which may directly solved by means of the classical simplex routine.

The optimal solution, obtained after six iterations, is given by :

$$g = -5{,}280{,}344.9$$

$$y_1 = 4.53743 \; ; \quad y_2 = 10.88983 \; ; \quad y_3 = 272.24584 \qquad (6)$$

$$y_4 = 42.19811 \; ; \quad y_5 = 31.76202$$

which in terms of the original variables gives f = - 5,280,344.9

$$x_1 = 4.53743 \; ; \quad x_2 = 2.40000 \; ; \quad x_3 = 60.00000 \qquad (7)$$

$$x_4 = 9.30000 \; ; \quad x_5 = 7.00000.$$

The solution of the original problem by means of non-linear programming methods |4, 5| lead, after a lot of iterations, to values of f which are 2 or 3 % below the true optimum but, in some cases, with value of the variable x_3 which is about 50 % erroneous.

In |6|, the authors show how a suitable choice of the behaviour model for a complex structural design - indeterminate prestressed bridges - leads to a benefit similar to that obtained by MYLANDER.

The idealization of the problem is based on an approach with sensitivity coefficients, as that proposed by GURUJEE |7|, and on a variable transformation; it is then allowed to solve this complex design problem by means of linear programming, without the actual problem be denaturated and taking account of all the technological requirements (cover thickness, anchorage dimensions, redundant effects of prestressing, friction losses, anchorage slippage,...). After the variable transformation, the problem remains partially non-linear but the authors have shown in |8| that the non-linear term, being of the order of 1 % with respect to its corresponding linear component, may be neglected in practice.

The authors would like to conclude by saying that for optimum design, as for all the other engineering activities, mathematics are a good servant but a bad master.

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SUMMARY

In structural optimization problems, it is nearly always observed that, in the search for a realistic solution, the suitability of idealization is more important than the choice of the solving algorithm.

RESUME

Dans les problèmes de dimensionnement optimal, il est généralement constaté que la recherche d'une solution réaliste dépend davantage de l'idéalisation du problème que du choix de l'algorithme de résolution.

ZUSAMMENFASSUNG

Bei der Optimierung von Tragwerken wird allgemein festgestellt, dass die Suche nach einer realistischen Lösung mehr von der Idealisierung des Problems als von der Auswahl des Lösungsalgorithmus abhängt.

Minimum Weight Plastic Design of Regular Rectangular Plane Frames

Calcul plastique pour un poids minimum de cadres plans rectangulaires

Plastische Bemessung auf Minimalgewicht für rechteckige, ebene Rahmen

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1. INTRODUCTION

Structural engineers have been concerned more with practical computational techniques for optimum structural designs than with theoretical results. On the other hand, scientific investigations on optimality criteria and optimal structures have been carried out mostly by researchers in the field of structural or applied mechanics. These two approaches are mutually compensating in order to develop more rational methods of structural designs.

The introductory report by A.B.Templeman has been primarily concerned with the hierarchy of optimum structural design problems and the corresponding computational techniques. Reference is made in his report to the linear theory of minimum weight plastic design and to the advantage of linear programming. It should also be recognized that the theoretical results on optimality criteria and optimal structures not only have the scientific significance but also lay the foundations and stimulate new ideas for developing practical computational techniques.

The purpose of this discussion is to call attention to the recent results [2-10] by the author and his colleagues on some general solutions derived analytically in closed forms to the problems of minimum weight plastic design of regular rectangular plane frames of practical interest and then to point out the theoretical and practical significances of those solutions.

2. FRAME MOMENT FOR REGULAR RECTANGULAR FRAME

Fig.1 shows a regular rectangular plane frame and one set of vertical and lateral design loads. The geometrical regularity in such a frame not only is reflected in design loads but also characterizes its structural behaviors and optimal plastic designs. In many practical design problems, the story shear resultant increases rapidly from the top floor toward lower stories as compared with the variation of vertical gravity loads. In those countries where frames must withstand against strong-motion earthquakes and strong gusts, *fairly large* lateral design loads are assigned. Under these circumstances, the first step of analytical treatment of a Foulkes-type problem is to assume an extremely deteriorated collapse mecha-

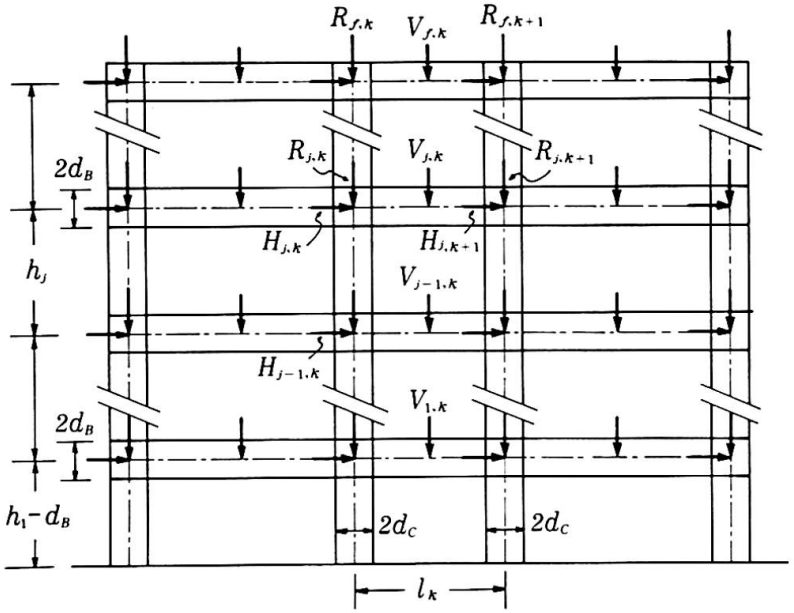

Fig.1
Regular
rectangular
frame

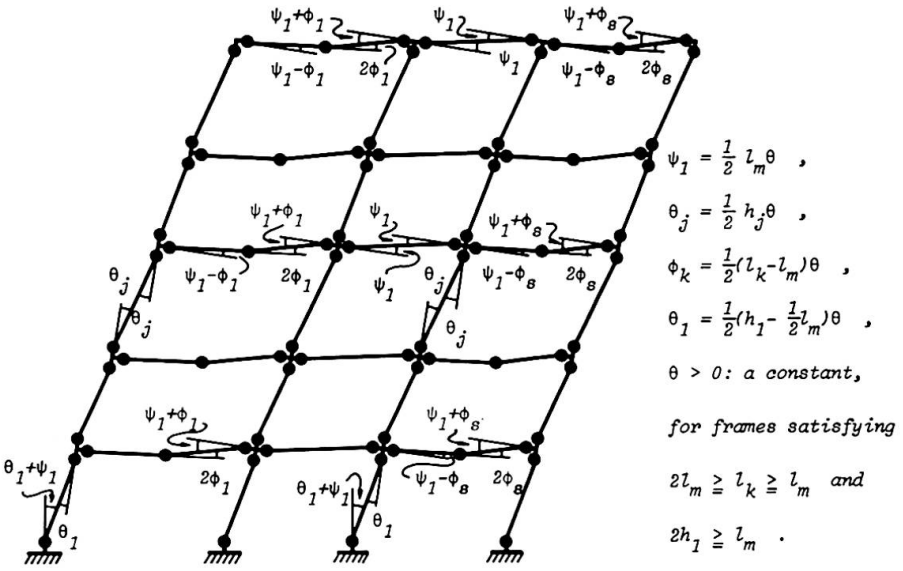

Fig.2
A Foulkes
mechanism
for
regular
rectangular
frame

nism shown in Fig.2, in which *simple* plastic hinges have formed at almost all the potentially critical sections except at the midspan sections of some particular bay(s) to be found as a part of the solution. Fig.3 shows that the corresponding bending moment diagram at plastic collapse may be conceived as the result of two-fold superpositions of decomposed diagrams. Each decomposed diagram is such that the moment equilibrium is maintained at the four corners with the same absolute value in the manner shown in Fig.3. This equal corner moment associated with this elementary moment diagram is called a "frame moment". A restricted minimization may then be carried out analytically in terms of the frame moments, and some statical conditions are derived under which the assumed bending moment diagram corresponds indeed to a general solution. It is then shown that the Foulkes mechanism condition can also be satisfied for a class of frames satisfying two simple geometrical conditions as shown in Fig.2.

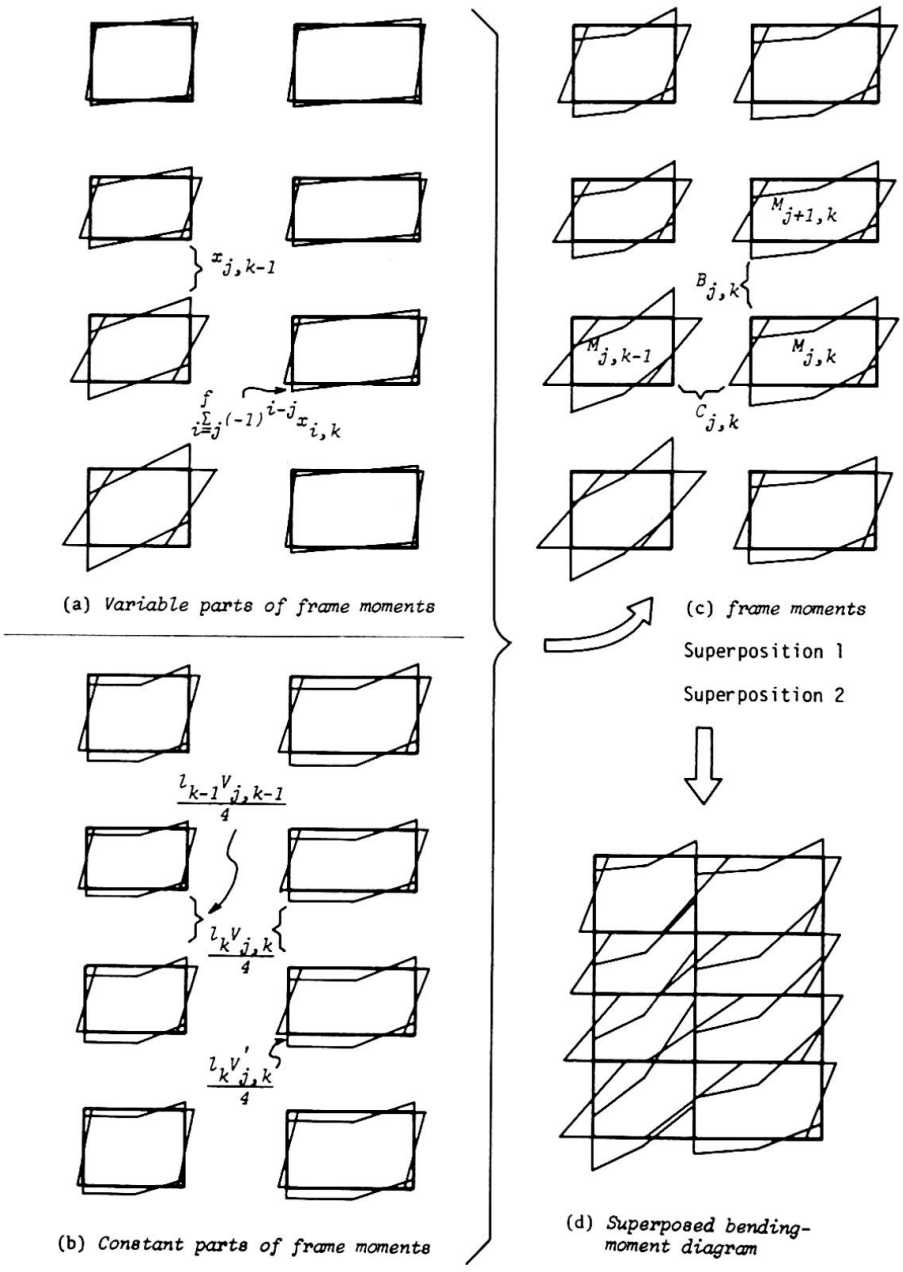

Fig.3 Frame moments and two-fold superposition procedure

3. APPLICATION OF FRAME MOMENT DECOMPOSITION TECHNIQUE

The two general solutions mutually exclusive and compensating on a design chart [2] not only clarify the general features of the classes of the minimum weight designs, but also provide a basis on which some modified general solutions can be derived to problems formulated more realistically by incorporating the axial force-bending moment interaction yield conditions for idealized beams and columns [3] shown in Fig.4. Fig.5 shows a part of modified Foulkes mechanism in a theory [4] in which only the idealized columns are required to satisfy the interaction yield condition shown in Fig.4. The regularity in the frame geometry enables one again to derive the general solutions and the statical and geometrical conditions analytically in closed forms [3, 4].

For the problem where reaction constraints have been incorporated within the framework of Foulkes' theory, a *bay shear distribution law* has been derived in [5] also on the basis of the concept of the frame moment and of the afore-mentioned two-fold superposition procedure.

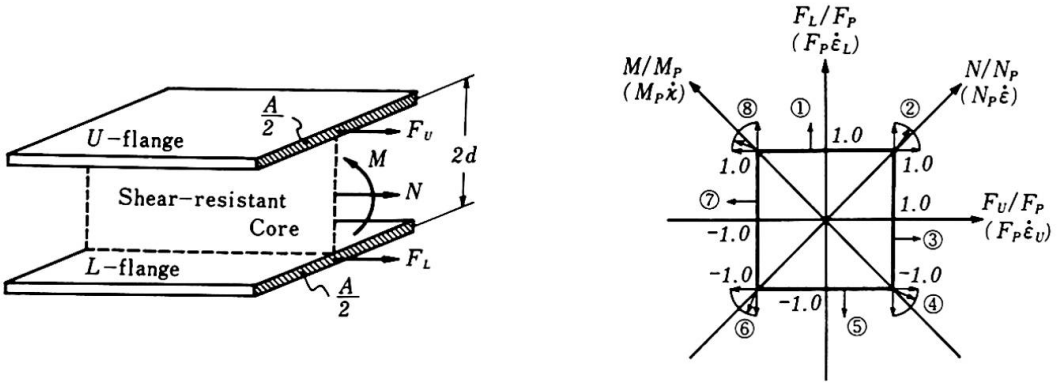

Fig.4 Axial Force-Bending Moment Interaction Yield Condition for an Idealized Sandwich Member

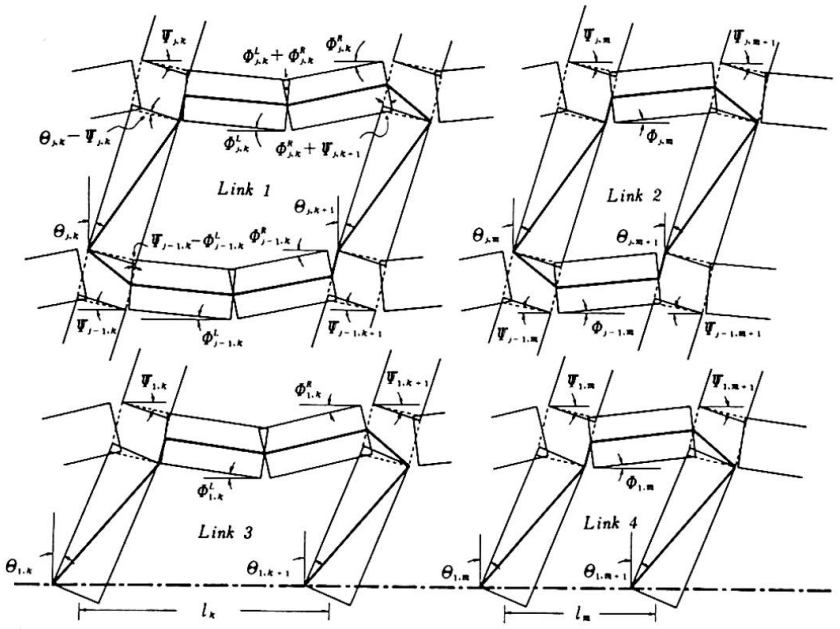

Fig.5 A modified Foulkes mechanism

For the problem of multi-story multi-span frames to be designed for five sets of design loads, a kinematical restricted maximization procedure has been developed in [6] by combining the primal-dual method of LP with a semi-inverse approach similar to [2]. Some general solutions have thereby been derived analytically in closed forms. Fig.6 shows a portal frame obeying an idealized interaction yield condition and subjected to two sets of design loads. Fig.7 shows a fundamental design chart for this frame. This chart together with the theory in [7] constitutes the foundation for a possible analytical attempt of incorporating the result of [3] and [4] in [6].

4. SIGNIFICANCE OF THE CLOSED FORM GENERAL SOLUTIONS

The theoretical significance of these general solutions are now obvious. Each general solution provides a basis for developing practically useful general solutions to problems of more realistic formulations, though some modifications may become necessary for the topmost few stories. The afore-mentioned results may be said to provide ample grounds for the fruitfulness of this successive refining process.

For practical application, these solutions must first be modified for the effect of inelastic stability and member design requirements. The author and his colleagues have already clarified to a certain extent through numerical large-deflection analyses that minimum weight frames can indeed withstand against static al-

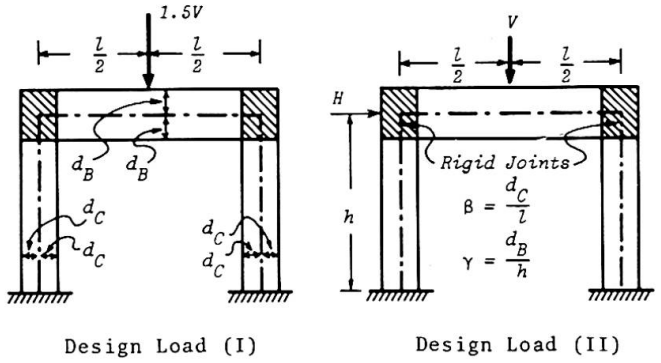

Fig.6
Portal frame
and
the two design
loads

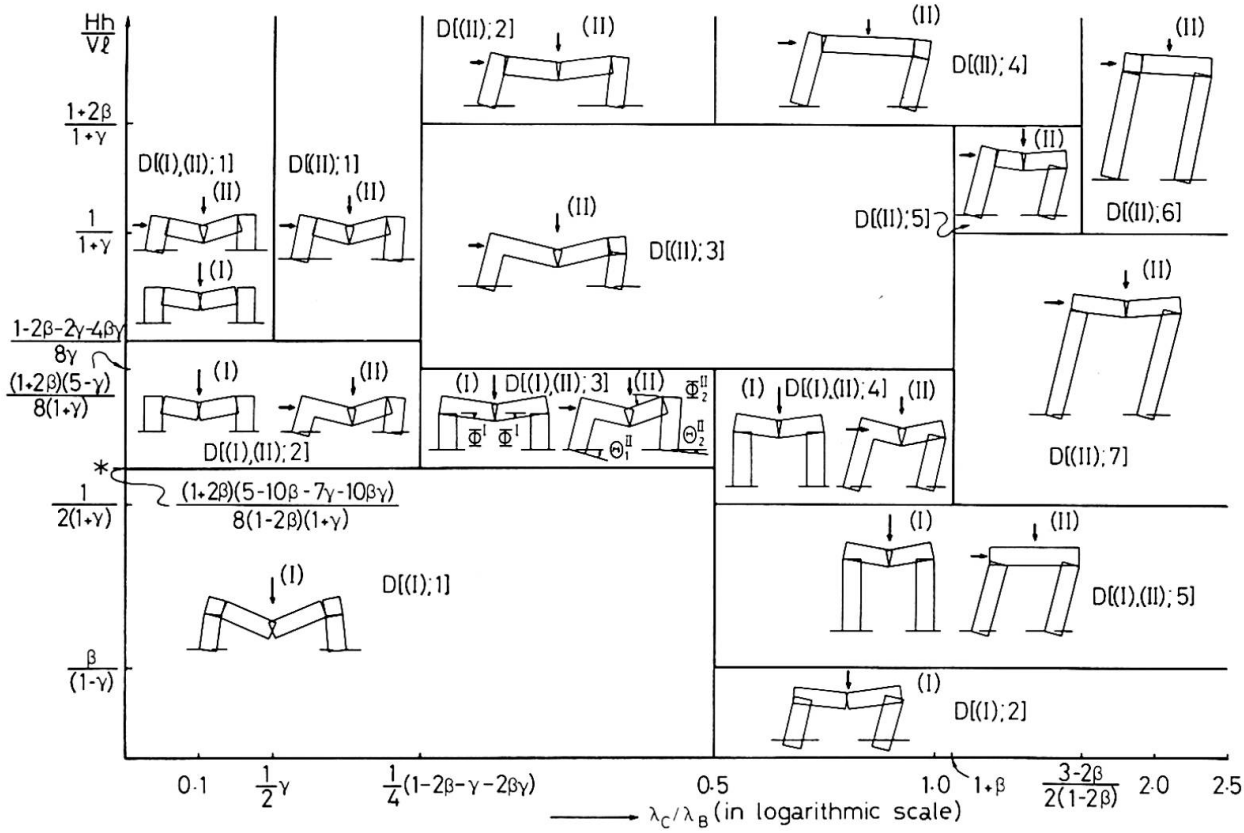

Fig.7 Design chart for a portal frame subjected to two sets of design loads

ternating lateral loads [8] and strong-motion earthquake disturbances [9] and are not particularly imperfection sensitive [10]. It should also be noted that the afore-mentioned solutions are the necessary consequences of the one-sided optimization using an approximate "failure" design criterion aside from the "serviceability" design criterion to be satisfied in practice.

Yet it can be said that the afore-mentioned solutions have the following significances: (i) they clarify the intrinsic features of the minimum weight plastic designs of regular rectangular frames at various levels considerably well; (ii) they will provide good initial solutions, if properly incorporated in a program, to start a numerical search for an optimal solution under additional constraints and may also be utilized as some standards for program verification. (iii) It may be well expected that the closed form solutions will be useful for seeking for optimum span length combinations analytically.

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SUMMARY

The frame moment decomposition technique due to the author and its applications to several more realistically formulated problems have been briefly described. The theoretical and practical significances of the analytical approach to the minimum weight plastic design problems have been explained in reference to the papers by the author.

RESUME

On décrit la méthode de décomposition des moments du cadre, proposée par l'auteur, et ses applications pratiques. La valeur théorique et pratique de cette méthode de calcul plastique, pour un poids minimum, est discutée.

ZUSAMMENFASSUNG

Die vom Autor entwickelte Methode der Momentenzerlegung sowie deren praktische Anwendungsmöglichkeiten werden beschrieben. Der theoretische und praktische Wert dieser Methode der plastischen Bemessung auf Minimalgewicht wird untersucht.

Structural Optimization via Penalty Methods: A New Type of Penalty Function

L'optimisation structurale par les méthodes de pénalisation: un nouveau type de fonction de pénalité

Optimierung von Tragwerken durch Strafmethoden: ein neuer Typ von Straffunktionen

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1. Introduction

Sound mathematical idealizations of practical design problems lead as a rule to highly nonlinear, and possibly nonconvex, programming problems.

The main effort in the field of computerized design methods should therefore be concentrated upon the implementation of versatile numerical procedures capable of solving, at least in principle, general mathematical programming problems. It is obvious that particular problems can be solved more cheaply by means of 'ad hoc' techniques exploiting their special properties, but it is the authors' opinion that the general approach should yield the major improvements to structural optimization, at the present stage of its development.

In this note, the attention is focussed on sequential unconstrained minimization techniques, which seem to be among the most interesting approaches for general automated design routines. A new kind of penalty function is introduced, and applied to a typical design problem, with the aim of assessing its capabilities.

2. Mathematical formulation

We consider the following type of problem

$$\begin{aligned} &\text{minimize} && f(x_i) \\ &\text{subject to} && g_j(x_i) \leq 0 \end{aligned} \qquad (i=1, \ldots, n;\ j=1, \ldots, m) \qquad (1)$$

From problem (1) the following parametric problem is derived

$$\text{minimize} \qquad f(x_i) + \sum_{j=1}^{m} \langle 1 + g_j(x_i) \rangle^{\alpha} \tag{2}$$

where the symbol $\langle \cdot \rangle$ has the meaning

$$\langle \cdot \rangle = \max (0, \cdot)$$

and the parameter α ranges over the open interval $(1, +\infty)$.

Each inequality constraint $g \leq 0$ is accounted for by a penalty term

$$p(g) = \langle 1+g \rangle^{\alpha} \tag{3}$$

From fig. 1 it is apparent that function (3) is neither an interior nor an exterior penalty function.

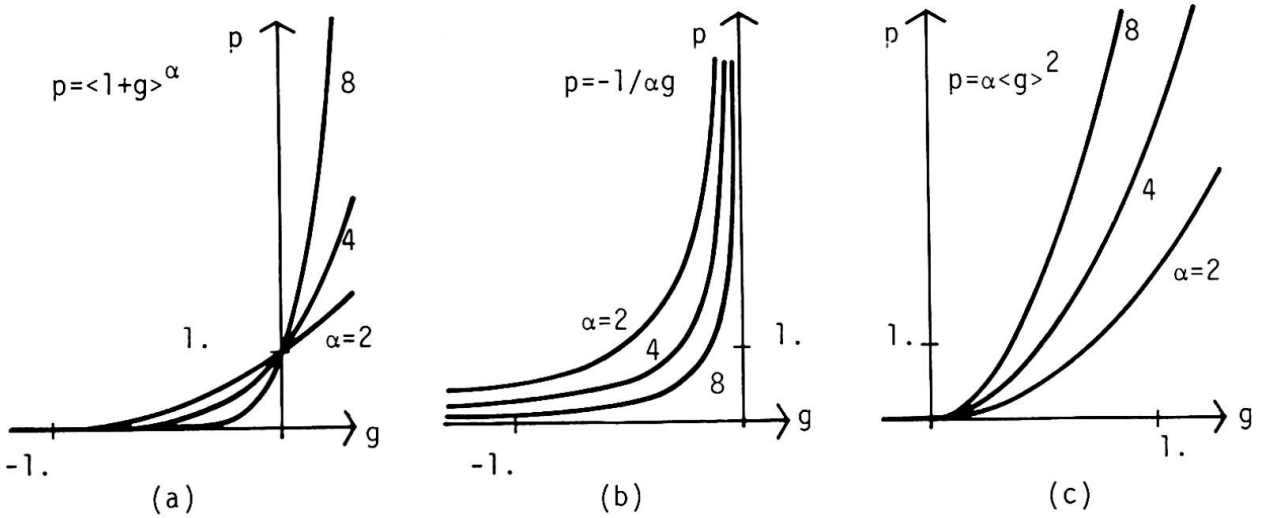

Fig. 1: Proposed penalty function (a) versus interior (b) and exterior (c) penalty functions.

The main properties of formulation (2) may be stated as follows:

i) if problem (1) has a (local) solution, a solution of problem (2) will approach it, when α approaches infinity;

ii) in contrast with interior penalty functions, penalty function (3) is defined over the range $-\infty < g < +\infty$;

iii) in contrast with exterior formulations, formulation (2) yields feasible minima for sufficiently large values of α, i.e. the solution of problem (1) is approached from the inside of its feasible region.

Properties ii) and iii) give an obvious advantage to penalty function (3) over interior and exterior penalty functions, respectively.

3. Allowable stress design of a truss with assigned topology

If a minimum weight design is sought, the objective function is easily ex

pressed in terms of member cross-sectional areas and joint coordinates. For each member and each load condition, the following constraints are considered

$$g_+ = \sigma/\sigma^+ - 1 \leq 0$$
$$g_- = \sigma/\sigma^- - 1 \leq 0 \qquad (4)$$

where σ^+ (σ^-) is the allowable tension (compression) stress of the considered member. If member buckling is accounted for, the compression limit σ^- depends on the (minimum) radius of gyration of the member cross-section. For a given type of cross-section, the radius of gyration can usefully be expressed as a function of the area, thus leaving only one design variable for each member. A second set of constraints will impose a minimum admissible value to each area. Displacement constraints may be obviously included.

The major task is to compute the stress and its gradient (the displacement method of analysis is of course preferable). Special attention must be devoted to the fact that stress constraints (4) are not defined over the entire design space: in fact, there exist (unfeasible) designs for which in one or more members the stress grows to infinity. This difficulty can be cured by introducing suitable modifications of the stress constraints (4) outside the feasible region, and by adopting a careful minimization strategy.

4. Numerical results

An algorithm (AUDE) for the numerical solution of automated design problems, based on the described formulation, has been developed. The minimization (2) is performed, for a sequence of suitably increasing (integer) values of α, using the Davidon-Fletcher-Powell method. Two-point cubic fit for successive unidirectional searches is used. Size and geometry variables are treated simultaneously.

The results obtained for a sample design problem, relative to a steel planar truss, are represented in fig.2. The lower chord is assumed to be straight and made up of six bars, long 5 m each. The total span of the upper chord is 30 m also, but its shape is free. All members are tubular, and their thickness is supposed to be adequately represented by the relationship

$$t = 1.5 + 0.02\ D \qquad (t,D \text{ in mm})$$

D being the diameter. Load conditions are specified by a single 10,000 kg concentrated load, moving along the lower chord. The allowable tension stress is assigned a value of 2,400 Kg/cm^2, and the allowable compression stress is computed in terms of the member slenderness ratio according to the Italian Code requirements.

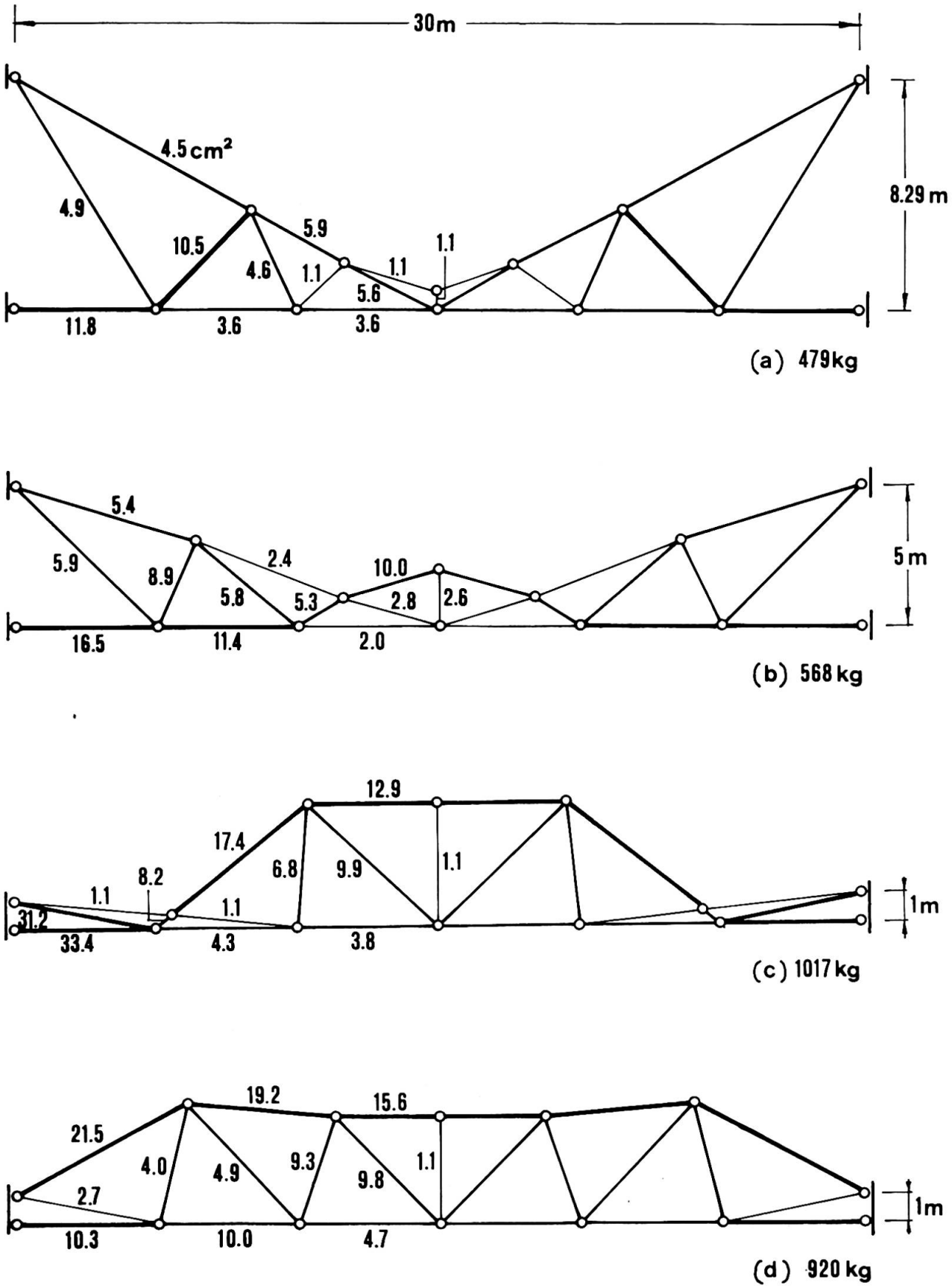

Fig. 2: Truss design

The lower chord should not undergo vertical displacements greater than 1/800 of the span. The truss should be designed for minimum weight.

Taking into account the obvious symmetry of the optimal solution(s), the above stated problem can be treated with 12 size variables, 6 geometry variables, and 3 load conditions. The optimal design obtained by AUDE is depicted in fig. 2a, where the member areas (in cm^2) are also reported. Note that 1.1 cm^2 was the minimum allowable area used in the computation. The weight of the optimum truss is 479 Kg, its height 8.29 m.

If now the distance H between the supports if given a fixed value, the geometry variables reduce to 5, and the optimum weight should obviously increase. Fig. 2b shows the solution obtained for H = 5 m. For H = 1 m two local optima have been detected (figs. 2c, d), the second one being a good candidate for the global solution.

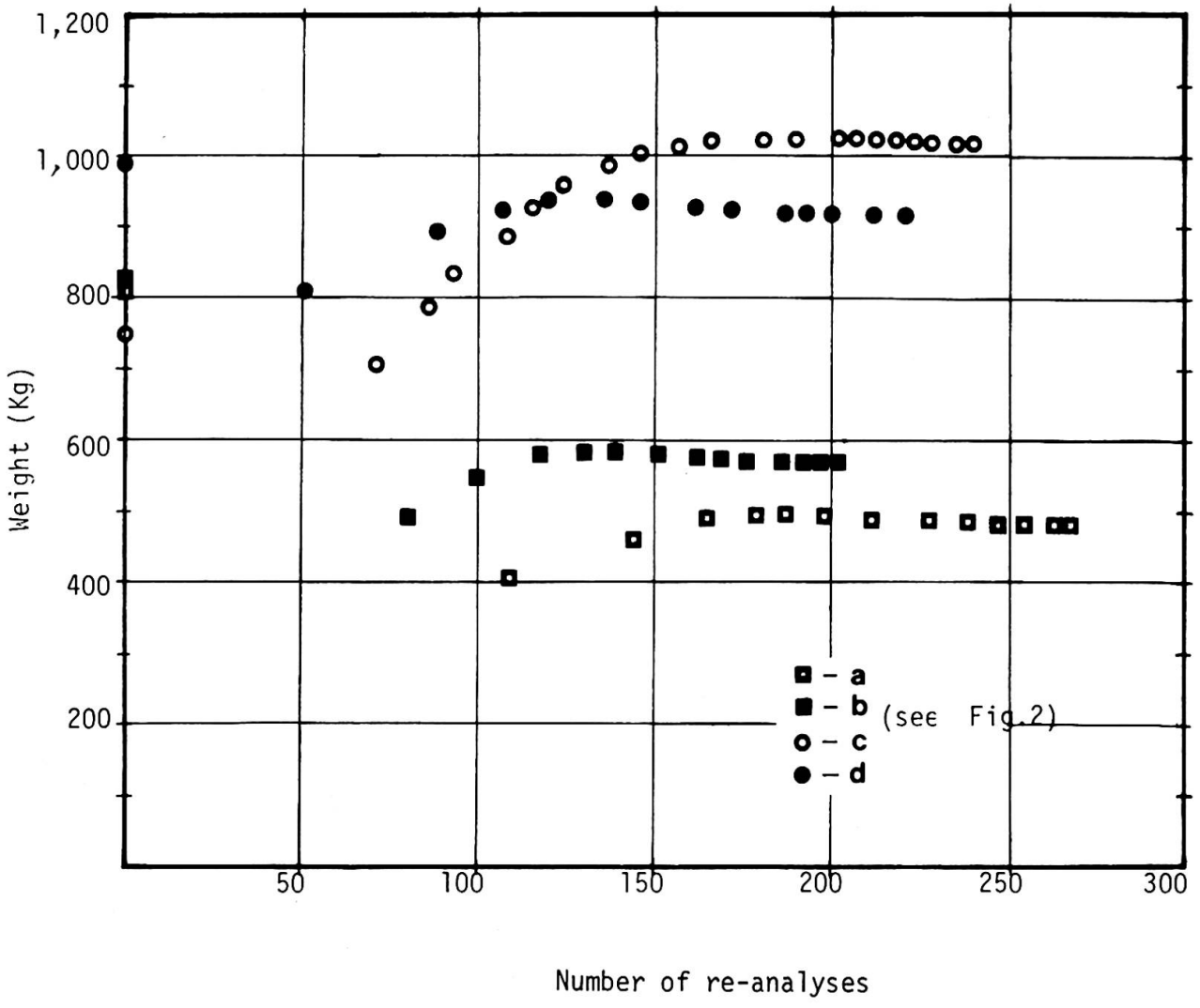

Fig. 3: Minimization trend

In each of these calculations, the parameter α was increased until a value of about 8000, and 200 ÷ 250 re-analyses were performed. Fig. 3 shows the sequences of minima relative to the four cases of fig. 2. As it is seen, after a drastic change on the first response surface, the objective function approaches rather smoothly its asymptotic value.

SUMMARY

An exponential penalty function is introduced and applied to a typical nonlinear and nonconvex design problem. Some results on geometry optimization of plane trusses are presented and discussed.

RESUME

On introduit une fonction de pénalisation exponentielle, et on l'applique à un problème typiquement non linéaire et non convexe d'optimisation structurale. On présente et on discute quelques résultats relatifs à l'optimisation géométrique de structures réticulées planes.

ZUSAMMENFASSUNG

Eine exponentielle Straffunktion wird auf ein typisch nichtlineares und nichtkonvexes Tragwerksproblem angewandt. Einige Ergebnisse über die Optimierung der Geometrie von ebenen Fachwerken werden angegeben und besprochen.

Über die Grundlagen und Methoden der Optimierung

On the Fundamentals and Methods of Optimization

Sur les principes et les méthodes d'optimisation

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1. Einleitung

Die analytischen Methoden des konstruktiven Ingenieurbaus sind in den letzten Jahren durch verstärkten Einsatz von Digitalrechnern mehr und mehr verfeinert worden. Das dabei erreichte hohe Niveau ist die Folge einer langen Tradition und einer daraus resultierenden Formalisierung der Berechnungsverfahren. Abgesehen von weiteren - sicherlich wichtigen - Verbesserungen (Berücksichtigung nichtlinearen Werkstoffverhaltens und großer Verformungen) sind jedoch keine grundlegend neuen Erkenntnisse mehr zu erwarten. Anders liegen die Verhältnisse bei der Tragwerkssynthese, bei der die Charakteristika des Bauwerkes als Unbekannte betrachtet und im Hinblick auf ein Bewertungskriterium unter Beachtung von technologisch - mechanischen Restriktionen festgelegt oder optimiert werden. Hier befindet man sich erst am Anfang einer Entwicklung, die sicherlich auch Rückwirkung auf die Tragwerksanalyse selbst haben wird.

Innerhalb der Synthese kommt den Optimierungsmethoden tragende Bedeutung zu, da erst mit ihrer Hilfe Syntheseprobleme zu lösen sind. Die Vielzahl der Optimierungen, die nur unzureichend die spezifischen Belange der Ingenieurpraxis berücksichtigen, gibt Veranlassung - in Ergänzung zum Aufsatz von TEMPLEMANN [E, S. 46 ff] einige kritische Anmerkungen zu machen.

2. Kritische Anmerkungen

Zwei Ursachen sind hauptsächlich dafür verantwortlich, daß die Optimierung zu zweifelhaften, da nur akademisch interessanten Lösungen führen kann:

- unrealistisch konzipierte Optimierungsmodelle,
- rein mathematisch orientierte Lösungsverfahren.

Im folgenden soll hierauf kurz eingegangen und einige Anregungen zur Überwindung einer Fehlentwicklung dargelegt werden.

2.1. Konzeption des Optimierungsmodells

Aus der Fülle akademischer Beispiele soll der beidseitig eingespannte Biegeträger minimalen Gewichtes als besonders typisches Beispiel dafür, daß die gefundene "Optimallösung" irrelevant ist, herausgegriffen werden (siehe Bild 1).

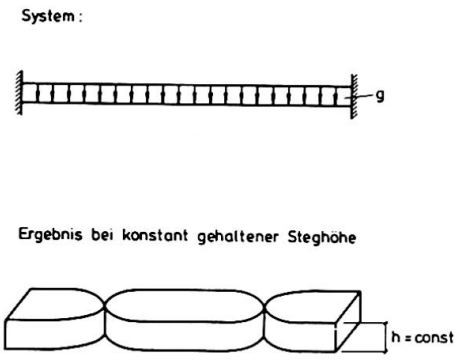

Bild 1: Optimierung des Gewichtes eines beidseitig eingespannten Trägers, ein "akademisches Beispiel" nach Hupfer (1)

Das gefundene Optimum ist die Folge eines schlecht konzipierten (ill - conditioned) Optimierungsmodells, das sich im Normalfall immer aus den drei Elementen

- Optimierungsvariablen
- Optimierungskriterium
- Optimierungsrestriktionen

zusammensetzt. Ist auch nur eine dieser Größen unzutreffend formuliert, zum Beispiel die Zielfunktion, verzerren sich die Optimierungsergebnisse. Viel weitreichendere Folgen stellen sich allerdings ein, wenn Restriktionen fehlen, fehlerbehaftet sind oder nicht in die Form gebracht werden können, wie es das später benutzte Lösungsverfahren verlangt, weil in diesem Fall der Lösungsraum ein völlig anderes Aussehen haben kann als in Wirklichkeit und somit andere Optima möglich sind. So hätte sich bei dem obengenannten Biegeträger mit Sicherheit eine andere Lösung ergeben, wenn technologische Restriktionen (Kontinuität der Kontur) berücksichtigt worden wären.

Will man praxisrelevante Optimierungsergebnisse sicherstellen, müssen alle drei Optimierungselemente ingenieurmäßig aufgebaut sein. Das ist aber im allgemeinen nur dann möglich, wenn man lediglich die Algorithmisierbarkeit der drei Elemente und nicht bestimmte mathematisch erzwungene Ausdrücke fordert.

2.2. Wahl des richtigen Lösungsverfahrens

Viele der vorgeschlagenen, heute gebräuchlichen Lösungsverfahren orientieren sich an der Denkart der Mathematiker, für jeweils eng abgegrenzte Problemklassen Lösungsmethoden zu erstellen, die nur unter bestimmten Voraussetzungen (hinsichtlich Konvexität, Stetigkeit, Differenzierbarkeit und bestimmter Formen der Nichtlinearität, etc.) anwendbar sind. Beispielsweise informiert das Bild

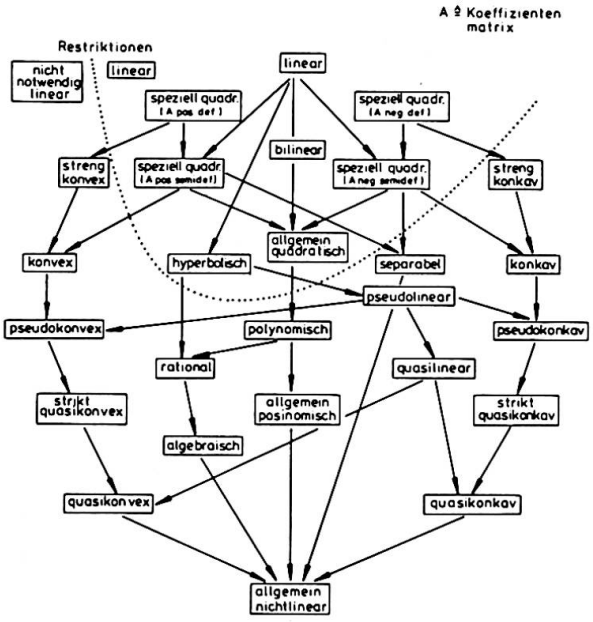

Bild 2:
Klassifikation nichtlinearer differenzierbarer Optimierungsaufgaben
nach Collatz . Wetterling (2)

2 über die vielfältigen Varianten allein bei differenzierbarer Zielfunktion. Im Bereich der technischen Optimierung gibt es jedoch nur selten Fälle, die derartig mathematisch klassifizierbar sind. Infolgedessen braucht der Ingenieur eine Art "Generallöser", der universell anwendbar, flexibel und effizient ist. Die Effizienz - ein bislang nicht eindeutig definierter Begriff - soll dabei u.a. danach beurteilt werden,

- wie groß der Arbeitsaufwand zur Anpassung des Lösungsverfahrens an ein beliebig nichtlineares Optimierungsproblem ist,
- wie groß der Arbeitsaufwand zur Anpassung der Zielfunktion und Restriktionen an das Lösungsverfahren ist,
- wie groß der Rechenzeitbedarf (CPU-time) bei bestimmten Testfunktionen ist,
- wie groß der Kernspeicherbedarf ist,
- wie das Konvergenzverhalten bei pathologischen Fällen ist.

Da insbesondere der Arbeitsaufwand zur Anpassung an Gegebenheiten erhebliche Kosten verursacht, muß der universellen Anwendbarkeit eines Lösungsverfahrens - unabhängig von der Klasse des Optimierungsproblems - größter Stellenwert eingeräumt werden. Die meisten der derzeitigen Lösungsverfahren genügen einer solchen Flexibilitätsforderung nicht. Die sogenannte Evolutionsstrategie dagegen [3], [4], die vom Verfasser mit Erfolg bei der Optimierung von Schalentragwerken eingesetzt wurde [4], [5], und im folgenden kurz vorgestellt werden soll, genügt dieser Forderung und kommt - bei entsprechender Weiterentwicklung - dem lang gesuchten "Generallöser" einen Schritt näher.

3. Evolutionsstrategie

Die Evolutionsstrategie ist ein sequentiell arbeitendes, iteratives stochastisches Suchverfahren mit Lernfähigkeit, bei dem die Suchschrittweite den Verhältnissen des jeweiligen Suchraumes angepaßt und selbst optimiert wird. Da die Suchschrittweite eine Zufallsvariable ist, kann man die Strategie als Monte-Carlo-Simulation höherer Stufe bezeichnen, deren Effizienz im Vergleich zu anderen Verfahren sich besonders bei vielen (ab 10 Variablen) bemerkbar macht. Sie darf aber auf keinen Fall mit der eigentlichen Monte-Carlo-Simulation verwechselt werden, weil ihr methodisches Vorgehen erheblich vom bekannten Monte-Carlo-Verfahren abweicht.

Das "Geheimnis" des Erfolgs und der Ausbaufähigkeit dieser Strategie ist darin begründet, daß die innere Logik des Verfahrens Optimierungsmechanismen der biologischen Vererbung, deren optimierender Effekt die Biologie tausendfach beweist, simuliert.

Durch eine einfache Konvergenzregel wird erreicht, daß optimale Fortschrittsgeschwindigkeit erzielt wird. Diese ist dann gegeben, wenn im Durchschnitt nach jeweils 5 zufälligen Suchvorgängen 1 Erfolg (Qualitätsverbesserung) erreicht wird. Andernfalls ist die Suchschrittweite (besser die Streuung der Suchschrittweite) zu vergrößern oder zu verkleinern. (Bild 3 zeigt die prinzipielle Arbeitsweise der Strategie an einem Beispiel).

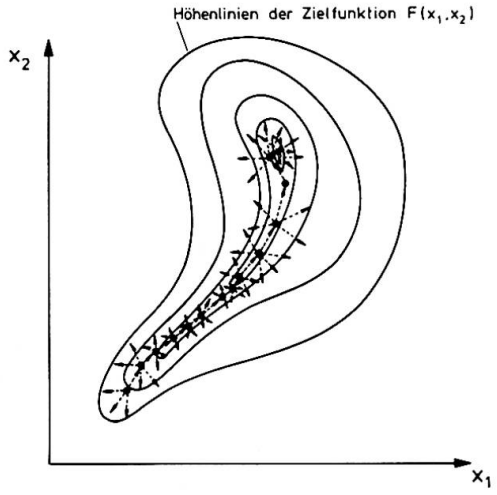

Bild 3:
Arbeitsweise der Evolutionsstrategie an einem zweidimensionalem Beispiel

4. Abschließende Bemerkung

Mit der Evolutionsstrategie besitzt der Ingenieur ein geeignetes Werkzeug, um Optimierungsprobleme im Konstruktiven Ingenieurbau zu lösen. Da die Routine in programmierter Form vorliegt und flexibel anwendbar ist, braucht sich der Benutzer nur noch um das problemabhängige Optimierungsmodell zu kümmern, eine Aufgabe, die jeder Ingenieur ohne große Mühe bewältigen kann.

SB

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ZUSAMMENFASSUNG

Es werden kritische Anmerkungen zum Aufbau eines auf die Praxis ausgerichteten Optimierungsmodells gemacht und die universell anwendbare Evolutionsstrategie wird als Lösungsverfahren technischer Optimierungsprobleme vorgestellt.

SUMMARY

Some critical remarks are made for the establishing of practical optimization models. Furthermore, a generally applicable solution method, the "evolution strategy" is proposed.

RESUME

Des considérations critiques sont faites pour l'établissement de modèles d'optimisation répondant aux besoins de la pratique. La méthode de la "stratégie évolutive" peut être utilisée de façon universelle pour résoudre des problèmes techniques.

Cora's Lesson

La leçon de Cora

Die Lehre von Cora

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1. INTRODUCTION.

Since 25 years we have been acquainted with the development of optimization-techniques for the design of structures. The first programs were mainly based on the "mechanism" approach of limit analysis.
After a general matrix theory for structures became available about 1966, design-programs based on the equilibrium method were developed [1].
Formulating a minimum-weight design, based on the equilibrium method, with only stress-limitations as constraint conditions, leads to a problem of linear programming (L.P.).
In practice, however, a design with only stress-limitations is not acceptable. The building codes require a number of additional constraint conditions, for instance with relation to stability and rotation capacity (fig. 3). Often these special conditions are strongly non-linear and they can be linearized only in the neighbourhood of a solution, so that one has to pass on to a non-linear programming technique.

2. CORA.

Recently a design program has been developed in the Netherlands (named CORA) for the design of braced steel frames. The designs meet the requirements by the national authority. Moreover, it is possible to distinguish between welded joint connections with or without stiffeners. Given an accepted input of geometry and loading (fig. 1), the program automatically produces the design of the frames using a sequential linear programming formulation (fig. 2). The introduction of the special constraint conditions means that a very large linear programming problem has to be solved a number of times. The solution has been found in applying a sophisticated L.P. algorithm and in the approximation of the constraint conditions by one plane with the aid of the method of least squares. Instead of five sets of constraint conditions (fig. 4) only one set has to be taken into consideration (fig. 5). It proved that 3 or 4 iterations were sufficient to obtain the theoretically exact values of the solution.

Problems arose from the wish to develop an instrument which will really be used in practice. This means that it should not be too expensive in use and that it should fit realistic structures. The more difficult problems however were formed by the codes themselves. In drawing up these codes the committee has had in mind of course a more or less sensible structural engineer and a proper structure. But in applying these rules and codes in an automated design program

irrevocably gaps and inconsistencies prove to be present in the codes. The computer is not a sensible structural engineer and he stumbles in the pitfalls caused by these gaps. Especially a mathematical optimization technique is a master in finding the inadmissible minima, as we noticed to our regrets several times.

3. THE LESSON.

The experience gained with this program has led us to the insight that if, in developing a design-process, one has to make allowance for requirements made by the government or a local authority, the design-program has to be separated from these prescriptions or codes. This applies to computer-aided design and the more so to computer-automated design. The reasons for this opinion - which we believe should be generally accepted - are:

a. The codes and prescriptions contain gaps and inconsistencies, which will always be recognized by the optimization-technique and unfortunately exploited.

b. The programmer who builds the design-program is not allowed to improve these inconsistencies.

c. By integrating the code into the design-program the program becomes dependent on this code. Codes have a temporary character. Adapting a design-program to code-changes will in general be very expensive or even impossible.

d. Working up codes into a form which is understandable by the computer is a lot of work which can best be left to those who draw up the codes instead of to every individual programmer.

4. CONCLUSION.

If - at least for the Building Industry - we want to leave behind us the more or less trivial examples, to proceed with our design-techniques to real life structures, we have to create the possibility to develop C.A.D.-programs that are independent from the codes. Therefore the codes have to be brought into a computer-readible form for instance in the way - indicated by Fenves [3] - by means of Decision Logic Tables. If this effort has led to success, the codes can be changed without consequences for the design-program (of course it will have consequences for the design itself).
Because the programs will be less vulnerable, software development for C.A.D. will become more popular. Two consequences seem to be of particular importance:

a. To develop C.A.D.-programs that can make codes of different countries accessible. This will make the software less dependent and less "national".

b. To study beforehand the effect of proposed code-changes, e.g. on economics or safety.

5. ACKNOWLEDGEMENT.

The CORA-program has been developed in the T.N.O. Institute for Building Materials and Structures, at the request of the Steel Structure Society and with financial aid of the C.I.A.D. (Computer Society of civil engineers). The first author was the chairman of the steering committee, the second author was responsible for the technical contents and the production. Most of the work was carried out by Ir. A.K. de Groot, Ing. G. Kusters and Ir. B. Speelpenning.

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SUMMARY

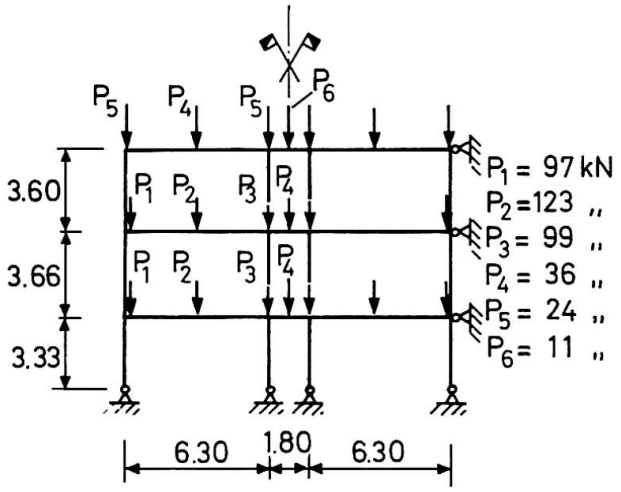
During the last three years a computerprogram (CORA) has been developed in the Netherlands for the design of braced steelframes. The frames designed with this program meet the requirements, made by the national authority. The building code requirements have been integrated into the design program, which has led to a very complex problem. The experience gained by solving this problem, has led to the insight, among other things, that design programs should be separated from rules and requirements in the building codes.

RESUME

Pendant les derniers trois ans un programme d'ordinateur (CORA) a été développé aux Pays-Bas pour le dimensionnement des ossatures en acier. Les ossatures dimensionnées avec ce programme satisfont aux conditions posées par les règles nationales pour les structures en acier. Les règles ont été insérées dans le programme de dimensionnement ce qui a conduit à un problème très compliqué. L'expérience a conduit à la conclusion que les programmes de dimensionnement doivent être séparées des conditions dans les règles ou codes.

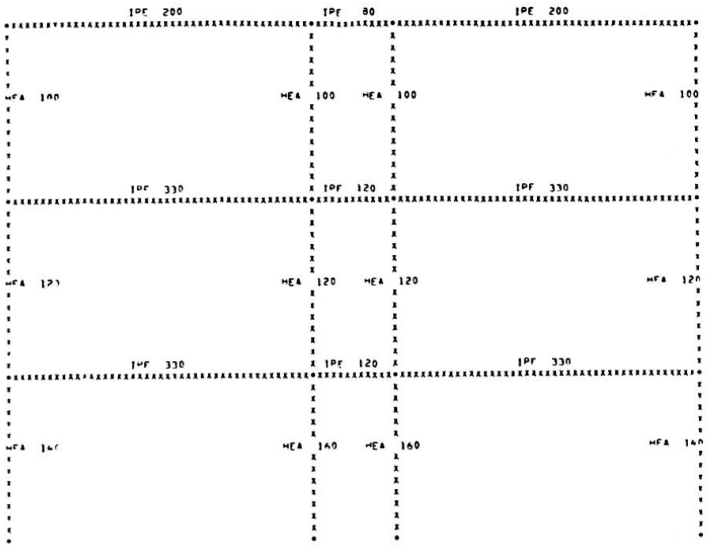
ZUSAMMENFASSUNG

Während der letzten drei Jahre ist in den Niederlanden ein Computerprogramm (CORA) für das Entwerfen von Rahmentragwerke aus Stahl entwickelt worden. Die Tragwerke die mit diesem Programm entworfen sind, erfüllen die Forderungen der nationalen Behörden. Die bautechnischen Anordnungen sind in das Entwurfsprogramm aufgenommen worden, was zu einem sehr komplizierten Problem geführt hat. Die Erfahrungen haben zu der Ansicht geführt, dass Entwurfsprogramme unabhängig und separat von Regeln und Forderungen in bautechnischen Anordnungen sein sollen.



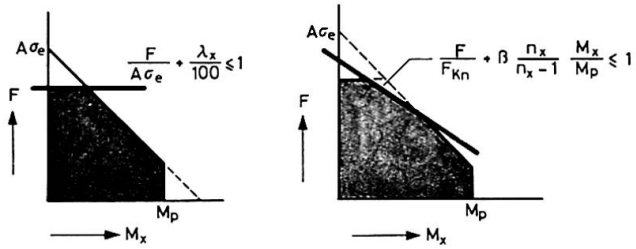
geometry and loading

fig.1



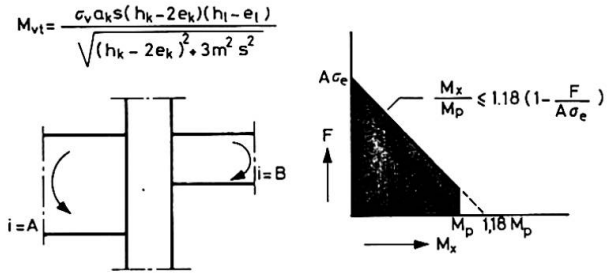
lineprinter plot

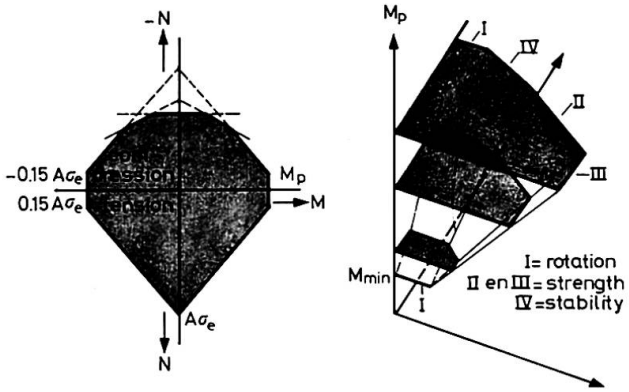
fig.2



constraints

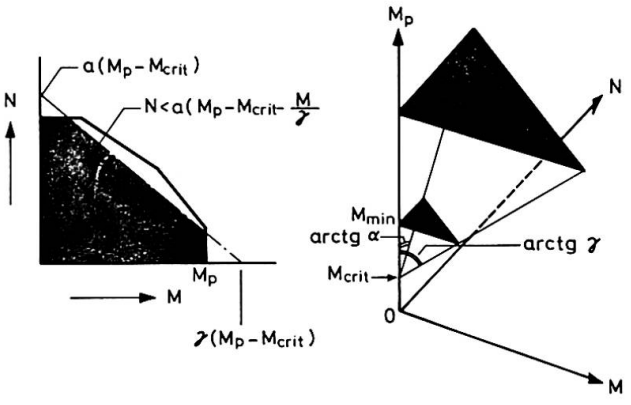
fig. 3





constraint surface

fig. 4



approximate constraint surface

fig.5

Basic Optimum Design Diagrams of Highway Plate Girders

Diagrammes fondamentaux pour le calcul optimum de ponts-poutres à âme pleine

Grunddiagramme für den optimalen Entwurf von Vollwand-Brückenträgern

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1. INTRODUCTION

In the structural system optimization process multistage systematic comparisons and decisions in such as behavior analysis, fabrication, erection, appearance etc. have to be made with regard to a number of optimized solutions of a wide variety of different structural schemes. For the progress of such a design process, the developments of the optimum design programs for specific structures are the essential efforts. The design approaches centered on this problem have been developed principally based upon the use of a combination of mathematical programming or rigorous numerical search techniques and structural analysis method. However the man-machine communication using these sophisticated optimization programs causes numerical complexity and large computer costs considerably in the large scale structural systems, accordingly, to avoid the difficulties and to make a realistic large scale system optimization process reliable, more practical and efficient design approaches have been expected [7,8].

On this problem a simplified optimum design process for minimum cost design of constant-depth highway plate girders is presented in this paper on the basis of the data manipulation of system optimum design diagrams developed by the graphical optimum design method [1,2] and the fundamental characteristics of the diagrams are discussed.

2. DESIGN CONDITIONS

The bridge types to be objective in this paper are 1st class simple span, 2 and 3 span continuous highway welded plate girders with constant web height throughout the bridge length. The range of bridge length, width, span ratio and the number of segments are indicated in Table 1. The arrangements of the main girders and thicknesses of reinforced concrete slabs are assumed as shown in Fig. 1 and Table 2 with due regard to the load distributions and appearances of the bridge superstructures.

The design variables to be determined are the steel type, M, all sectional dimensions, X, and the length, ℓ, to be used for each girder segment.

Design criteria imposed on the steel girder section are constraints on allowable stresses, plate thicknesses for stabilities of the girder and minimum rigidities of vertical and horizontal stiffeners, and which, as well as the displacement constraints of the girder, are taken from the "Specifications for Steel Highway Bridges" [5]. Discrete constraints on commercial availability of plate thicknesses are also considered. The steel types available for the design are assumed as SS41, SM50 and SM58 steels which physical and economical characteristics are described in the Specifications and the "Table of Prime Costs for Steel Highway Bridges" [6] respectively.

The total cost of the girder, TCOST, is assumed to consist of material cost, CM, fabrication cost, CFF, and welding cost, CWF, which are evaluated with reference to Ref. [6].

$$TCOST = \sum_{i=1}^{NS} COST_i \times \ell_i$$
$$= \sum_{i=1}^{NS} [CM_i(X,BP,SP,EX) + CFF_i(X,KS,CP) + CWF_i(X,WL,CP)] \times \ell_i \qquad (1)$$

in which NS = number of segments, COST = minimum cost per unit length of girder segment, BP = base price of steel plate per unit weight, SP = extra charge for steel type, EX = extra charge for plate thickness and width, KS = number of workmen to fabricate unit weight of steel plate except welding, CP = daily wage of a workman, WL = welding length in specific size. SP and the charge for pre-heating in welding vary with steel type used.

3. EFFECTS OF PRICE RATIOS

Since the optimum solution and the total cost of a girder with specific bridge length and width are complex functions of the design factors such as X,BP, EX,CP,KS, etc. as described in the previous section, it seems to be considerably difficult to presume the effects to the optimum solutions due to the changes of the design factors. However the ratio of CP to BP may be supposed to be the most effective factors. Consequently at first stage of this study the effects due to the changes of unit price ratios CP/BP to the optimum solutions are investigated.

Table 3 shows the examples of optimum solutions for the girders at various price ratios. As seen clearly in this table the optimum steel type, M, moment of inertia, I, and length, L, to be used for each girder segment are extremely insensitive to a wide variety of the unit price ratios except the case CP/BP=0.0 which is an unconsiderable case in the practical design problems. These phenomena can be seen at every span lengths and bridge types in the minimum cost design of highway plate girders and it has been cleared that the optimum design diagrams presented in the following sections may be applicable for a wide range of the ratios of CP to BP with sufficient accuracy.

4. SL(BL)-OPTIMUM WH RELATIONSHIPS

The minimum total cost of the plate girder with specific SL, BW and NS is changed concavely with web height on the whole, but locally several minimum solutions exist as noted in Ref. [1,2]. The optimum web height, WH_o, therefore should be decided by comparing the results in each discrete web

Table 1. Bridge Types, Lengths, Widths, Span Ratios and Segment Numbers

BRIDGE TYPE	BRIDGE LENGTH	BRIDGE WIDTH	SPAN RATIO	SEGMENT NUMBER
SIMPLE SPAN G.	20~40 m	6,7,8,9 m	——	5,7,9
2 SPAN CONT. G.	40~80	6,7,8,9	1.00:1.00	8,10,12,14
3 SPAN CONT. G.	60~120	6,7,8,9	1.00:1.00:1.00 1.00:1.30:1.00	13,15,23

(Unit = m)

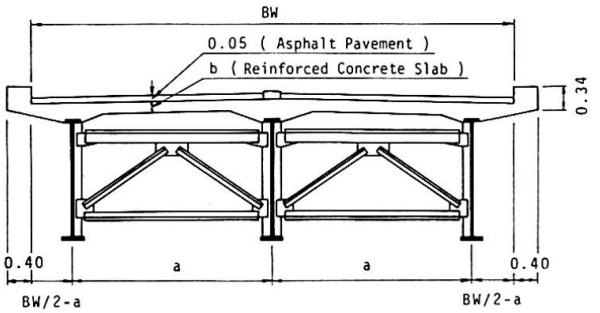


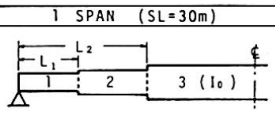
Fig. 1. Cross Section of Girder Bridge

Table 2. Girder Spaces (a) and Thicknesses of Reinforced Concrete Slabs (b)

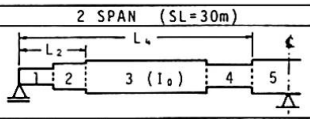
Bridge Width = BW (m)	6.00	7.00	8.00	9.00
a (m)	2.4	2.8	3.3	3.8
b (cm)	19.0	20.0	22.0	23.0

Table 3. Examples of Optimum Solutions for the Girders at various Price Ratios (BW = 8.00m , WH = 170cm)

1 SPAN (SL=30m)

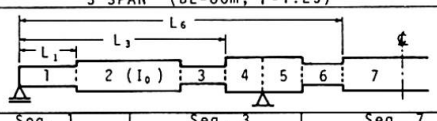


2 SPAN (SL=30m)



CP/BP	Seg. 1			Seg. 2			Seg. 2			Seg. 4		
	M_1	I_1/I_0	L_1/BL	M_2	I_2/I_0	L_2/BL	M_2	I_2/I_0	L_2/BL	M_4	I_4/I_0	L_4/BL
0.0	SS41	0.656	0.098	SM50	0.963	0.228	SS41	0.984	0.097	SS41	0.882	0.447
0.023	SM50	0.501	0.103	50	0.966	0.230	SM50	0.749	0.102	SM50	0.678	0.447
0.045	50	0.504	0.104	50	0.969	0.231	50	0.746	0.101	50	0.708	0.444
0.068	50	0.531	0.110	50	0.964	0.229	50	0.748	0.101	50	0.709	0.444
0.082	50	0.532	0.111	50	0.963	0.229	50	0.752	0.103	50	0.664	0.451
0.098	50	0.533	0.111	50	0.967	0.230	50	0.757	0.104	50	0.665	0.450
0.136	50	0.502	0.103	50	0.946	0.223	50	0.757	0.104	50	0.665	0.451

3 SPAN (BL=80m, η=1.25)



Seg. 1			Seg. 3			Seg. 7		
M_1	I_1/I_0	L_1/BL	M_3	I_3/I_0	L_3/BL	M_7	I_7/I_0	L_6/BL
SS41	0.564	0.043	SS41	0.725	0.273	SS41	0.969	0.424
SM50	0.566	0.043	SM50	0.755	0.270	SM50	0.976	0.429
50	0.566	0.043	50	0.755	0.270	50	0.976	0.429
50	0.566	0.043	50	0.756	0.270	50	0.977	0.429
50	0.560	0.042	50	0.753	0.269	50	0.974	0.431
50	0.566	0.043	50	0.756	0.270	50	0.977	0.429
50	0.566	0.043	50	0.755	0.270	50	0.978	0.429

BP = Base Price of Steel plate (yen/ton)
CP = Daily Wage of a Workman (yen/(person·day))
M = Opt. Steel Type
I = Opt. Moment of Inertia (cm^4)
L = Opt. Distance of variation of Sectional Dimensions from Left End Support

height. Fig. 2 shows WH_{opt} for every bridge types and bridge widths. They are changed stepwise and keep constant values for fairly wide ranges of span lengths. Discrete web plate thicknesses play an important role in the decision of optimum web heights and the ratios of web heights to thicknesses fairly coincide with the upper limits prescribed by the stability constraints of the girders. It is to be notable furthermore that only 170,200 and 230cm are selected almost as optimum web heights for the plate girders with nonuniform cross sections with a few exceptions of 190cm at the small ranges of 2, 3 span continuous girders.

5. OPTIMUM M, I, L, SDIM

Optimum M, I and L to be used for the segments of the girder with the optimum web height decided in the previous section may be determined efficiently by using graphical optimization algorithms [1,2]. In the algorithms M_{opt} and I_{opt} for each segment are determined by comparing the minimum costs of unit length of girder segment for every steel type namely SS41, SM50 and SM58 steels which are evaluated straightly with use of the maximum working bending moment, BM, and the relationships between moment of inertias and maximum resisting bending moments, RBM, and minimum costs, COST, of the girder sections which are provided from the suboptimization of girder elements. According as bending moment increases, higher strength

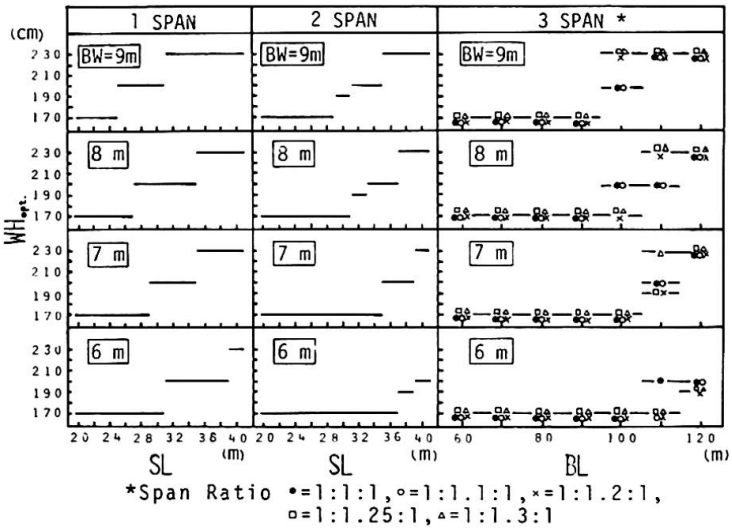


Fig. 2. SL(BL)-$WH_{opt.}$ Relationships for 1,2,3 Span Girders

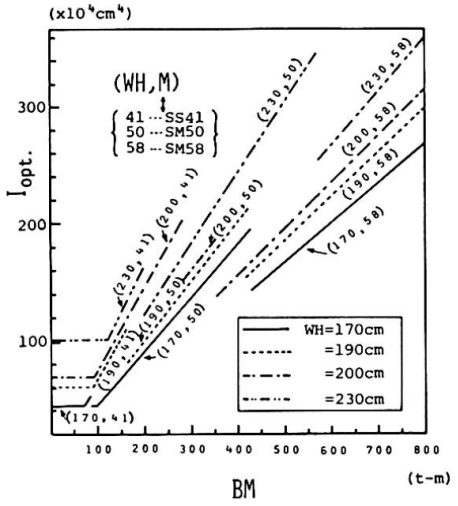


Fig. 3. BM-Opt. M and I Relationships (WH=170,190,200,230cm)

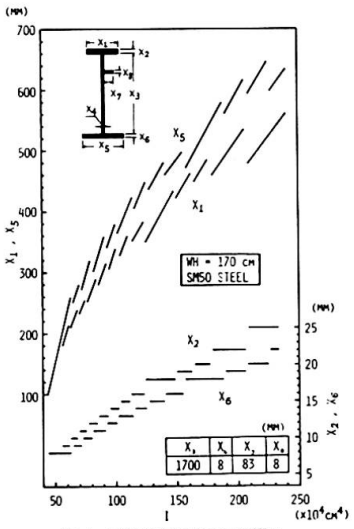


Fig. 4. I-SDIM Relationships for WH=170cm, SM50 Steel

Table 4. Opt. L_i/SL(BL) for the Bridge Types and Number of Segments

Type of Girder	Number of Seg.	Span Ratio		ξ (=L_i/BL) (Symmetric)
SIMPLE SPAN G.	5			0.104 0.238 0.500
	7			0.061 0.134 0.234 0.500
	9			0.053 0.118 0.194 0.298 0.500
2 SPAN CONT.G.	8			0.070 0.323 0.447 0.500
	10			0.069 0.330 0.434 0.468 0.500
	12			0.046 0.101 0.302 0.378 0.453 0.500
	14			0.046 0.101 0.333 0.431 0.457 0.478 0.500
3 SPAN CONT.G.		1:1.00:1	13	0.049 0.237 0.309 0.334 0.376 0.455 0.500
			15	0.031 0.067 0.233 0.305 0.334 0.384 0.463 0.500
		1:1.10:1	13	0.047 0.220 0.295 0.323 0.363 0.439 0.500
			15	0.030 0.066 0.216 0.291 0.323 0.367 0.466 0.500
		1:1.20:1	13	0.046 0.203 0.280 0.313 0.351 0.429 0.500
			15	0.029 0.064 0.198 0.279 0.313 0.349 0.434 0.500
		1:1.25:1	13	0.046 0.194 0.273 0.308 0.345 0.424 0.500
			15	0.028 0.062 0.192 0.272 0.308 0.344 0.428 0.500
			23	0.029 0.063 0.180 0.209 0.262 0.287 0.308 0.328 0.354 0.410 0.441 0.500
		1:1.30:1	13	0.042 0.188 0.263 0.303 0.340 0.420 0.500
			15	0.027 0.060 0.183 0.264 0.303 0.340 0.424 0.500

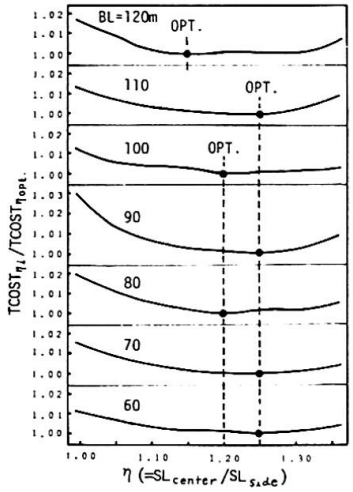


Fig. 5. Span Ratio-TCOST Ratio Relationships for 3 Span Continuous Girders

steels namely from SS41 to SM50 and SM58 are selected as optimum steel types and the corresponding optimum I are therefore varied discretely. The relationships between BM and optimum M, I for WH_{opt} = 170, 190, 200, 230cm are shown in Fig. 3. Horizontal lines of the relationships are caused by the constraints on minimum widths and thicknesses for flange plates. The arrangement of optimum steel types and moment of inertias of the girders therefore differ with values of the maximum bending moment diagrams. On the other hand, the ratios of optimum segment lengths do not change so much with span lengths and web heights, as tabulated in Table 4, they may be represented by a particular set of the ratios for a specific segment number and girder type with quite few exceptions. It has been cleared that the increments of total costs of the girders designed with these segment length ratios are less than 0.3% at most compared with the correct solutions. Optimum sectional dimensions, SDIM, of the girder segments can be decided directly from the moment of inertia and optimum sectional dimension diagrams for the web height and steel type as shown in Fig. 4 for WH=170cm and M=SM50 steel. Flange widths X_2 and X_6 are increased like as sawteeth with discrete changes of flange plate thicknesses as seen in the figure.

6. SPAN RATIO-TCOST RELATIONSHIPS OF 3 SPAN CONTINUOUS GIRDERS

For the 3 span continuous girder design, the variation in the minimum total cost with span ratio may be one of the interesting features and the relations for the girders with nonuniform cross sections are presented in Fig. 5. According to this result the most economical span ratio for plate girder superstructures with nonuniform cross sections is scarcely varied with bridge lengths and it may be decided as almost 1.20∼1.25. It should be emphasized also, however, the fact that the relative differences of total costs in the range of span ratio from 1.10 to 1.35 are considerably small as less than 0.7%. On the contrary, the optimum span ratio for the girder with uniform cross section reduces with bridge lengths from 1.10 to 0.95 for bridge length 60m∼120m and the relative differences of total costs are changed so much as the order of several percents.

7. ABM-OPTIMUM WH, M, I, SW RELATIONSHIPS FOR THE GIRDER WITH UNIFORM CROSS SECTION

Optimum solutions of the girders with uniform cross section may be decided directly such as example A or B in Fig. 6 by using the relationships between absolute maximum bending moment, ABM, and optimum design variables, WH, M, I and SW without reference to the bridge types. Different from the cases of girders with nonuniform cross sections, in which only one optimum solution is obtained for each span length, two optimum solutions, one for SM50 steel another for SM58 steel, which give the same minimum total cost are existed in a wide range of absolute maximum bending moments. Optimum web heights for the girder with uniform cross section differ with optimum steel types and they are decided almost as 170, 240, 270cm for SM50 steel and 200, 230, 260cm for SM58 steel.

8. DESIGN PROCEDURE USING OPTIMUM DESIGN DIAGRAMS

By using the design diagrams stated above as data banks, the minimum cost design of highway plate girders may be carried out simply, and the design procedures are shown in Fig. 7. The design process begins by choice of the segment number, NS, of the girder. The optimum web height and segment lengths are decided from SL(BL)-WH_{opt} and SL-L/BL relationships respectively. In the next place assumptions are made of moment of inertia and girder weights at each segment, the girder is then analyzed. The optimum steel type, moment of inertia and weight to resist the maximum bending moment at each segment are decided straightly from BM-Opt. M, I, SW relationships for the optimum web height. The girder analysis should be repeated until maximum bending moment, namely optimum M, I, SW, are converged. Optimum sectional dimensions for each segment are decided derectly from I-SDIM relationships for the web height and steel type. The total cost of the optimized girder is computed by using I-COST relationships or eq.(1). The entire looping is then carried out again for other NS if necessary. The girders with arbitrary web heights may be designed similarly without the selections of WH_{opt}.

9. CONCLUSIONS

This paper has dealt with fundamental optimum design diagrams effective to the minimum cost design of constant depth highway plate girders and an efficient systematic optimum design process based on the data manipulation of the design diagrams has been proposed.

The design diagrams described herein in detail are only that for the optimum web heights at each span length, but the diagrams for other considerable web heights have been developed as well. It has been confirmed also that the diagrams are applicable with accuracy to a wide range of unit price ratios of materials to workmen.

By using the optimum design diagrams as data banks, the minimum cost design of highway plate girders may be carried out by the simple

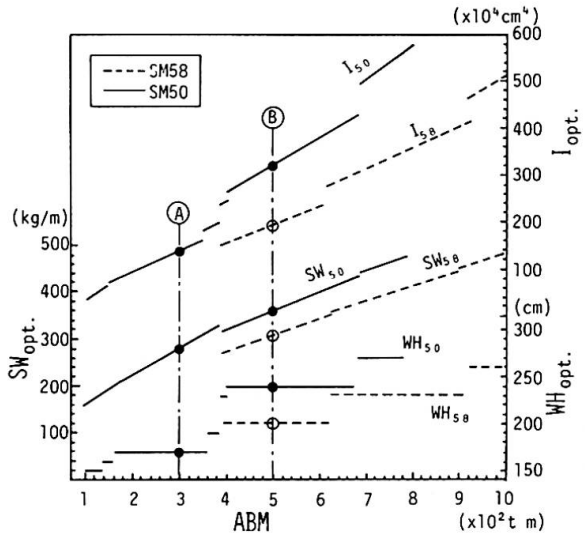


Fig. 6. ABM-Opt. WH,M,I,SW Relationships for the Girder with Uniform Cross Section

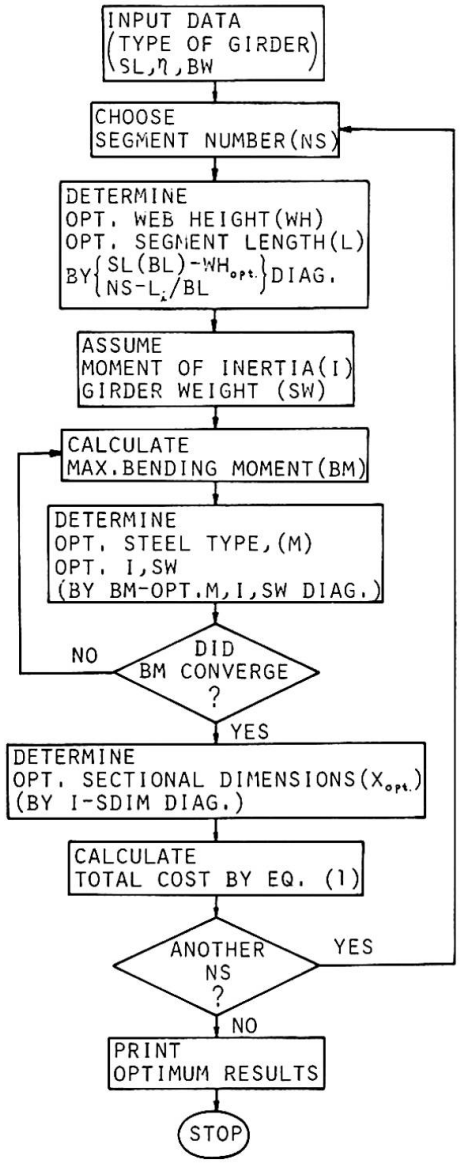


Fig. 7. Flow Chart of the Design Procedures

design procedures. The main computation required for the design is girder analysis only and most part of the design decisions can be made straightly from the diagrams.

The design procedures proposed in this paper may be utilized as one of the element design programs for specific structures in a general purpose system optimization program for highway bridges.

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SUMMARY

The basic optimum design diagrams are presented for the purpose of minimum cost design of 1 to 3 span highway plate girders; the fundamental characteristics of the diagrams are clarified. By using the design diagrams as data banks, minimum cost design of highway plate girders may be carried out in detail by a simple design procedure. The design procedure can be utilized as one of the element design programs in a general purpose system optimization program for highway bridges.

RESUME

On présente les diagrammes fondamentaux et leurs caractéristiques essentielles, pour un calcul, avec frais minimum, d'un pont-route à poutres à âme pleine de 1 à 3 travées. L'usage de ces diagrammes comme banque de données permet de faire le calcul détaillé selon un procédé simple, qui peut être utilisé comme un des programmes élémentaires d'un système général d'optimisation pour les ponts-routes.

ZUSAMMENFASSUNG

Grunddiagramme des optimalen Entwurfes werden mit ihren Charakteristiken dargestellt. Sie erlauben eine Berechnung mit Minimalkosten von 1 bis 3 feldrigen Strassenbrücken aus Stahl-Vollwandträgern. Diese als Datenbank verwendeten Diagramme gestatten ein Minimalkosten-Entwurf bis ins Detail. Das Verfahren lässt sich als eines der elementaren Entwurfsprogramme im allgemeinen System-Optimierungs-Programm für Strassenbrücken benutzen.

A Contribution to the Optimum Design of Prestressed Plane Cable Structures

Une contribution au calcul optimal de structures planes de câbles prétendues

Ein Beitrag zur Optimierung von ebenen vorgespannten Seiltragwerken

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A research for the optimum design of prestressed plane cable structures is here biefly referred.

The starting point are P. Pedersen's (1) and G.N. Vanderplaats and F. Moses's (2) papers, where trusses are optimized when cross-sectional areas and joint coordinates are the design variables.The techniques here used must be substantially changed to be applied to prestressed cable structures. This is due to initial prestress (the necessary prestress to grant the wanted design requirements) which is to be kept constant under different loading conditions while cross-sectional areas and the geometry of the structure change.This need really complicates the computational process since, keeping the initial prestress constant, big changes in bar forces correspond to small displacements in joints.

Two kinds of prestressed plane cable structures have been optimized: a standard scheme[1] [Fig. 1,a] and a new type of prestressed cable-stayed structure[2] [Fig. 1,b] whose good static and dynamic behaviour had been shown in a previous work (3).

Both types of structures satisfy the necessary and sufficient conditions for fully stressed design, when at least two load systems are considered. Two uniformly distributed loads, a downward load (p_{max}) and an upward load (p_{min}), are here considered besides prestressing.

The problems relating to prestressed cable structures are of non-linear type. Yet, linear theory has been used because the very many checks carried on by the non-linear theory (3),(4) showed that results from linear analysis are valid, at least as far as localization of good structural parameters is concerned.

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1. Henceforth indicated by TSHS (two surface hanging structure).

2. Abbreviated by PCSS .

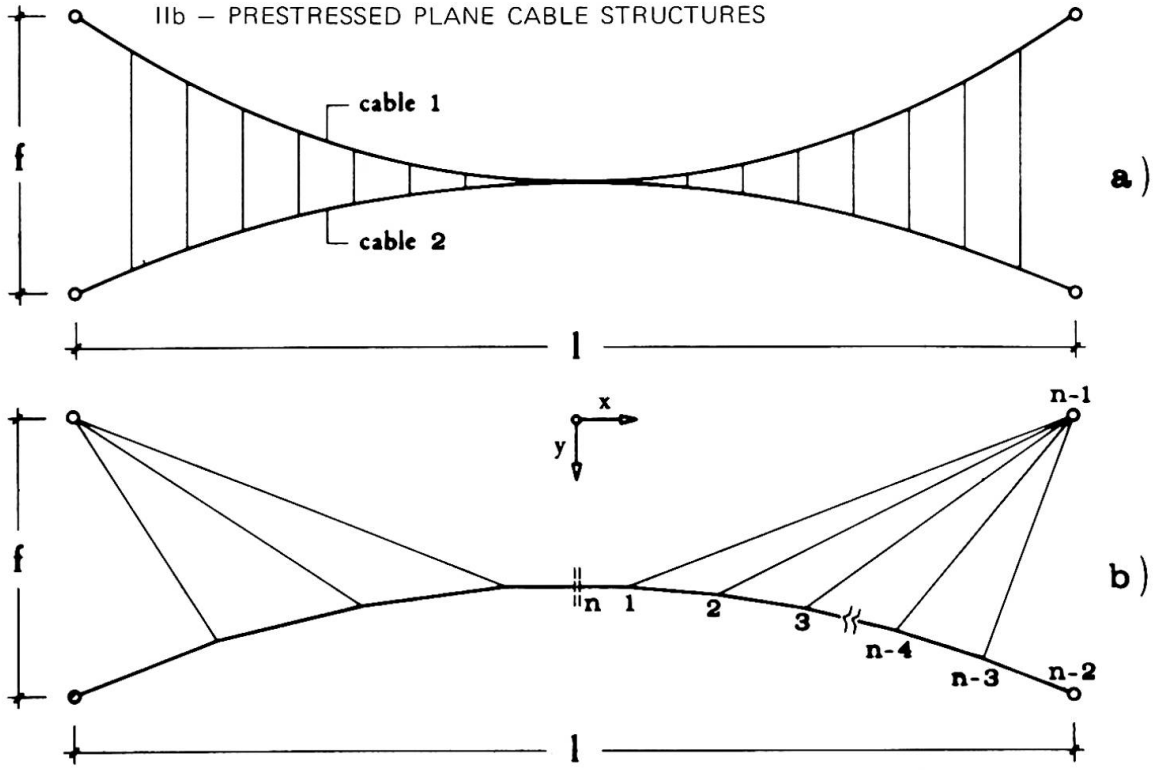

Fig. 1

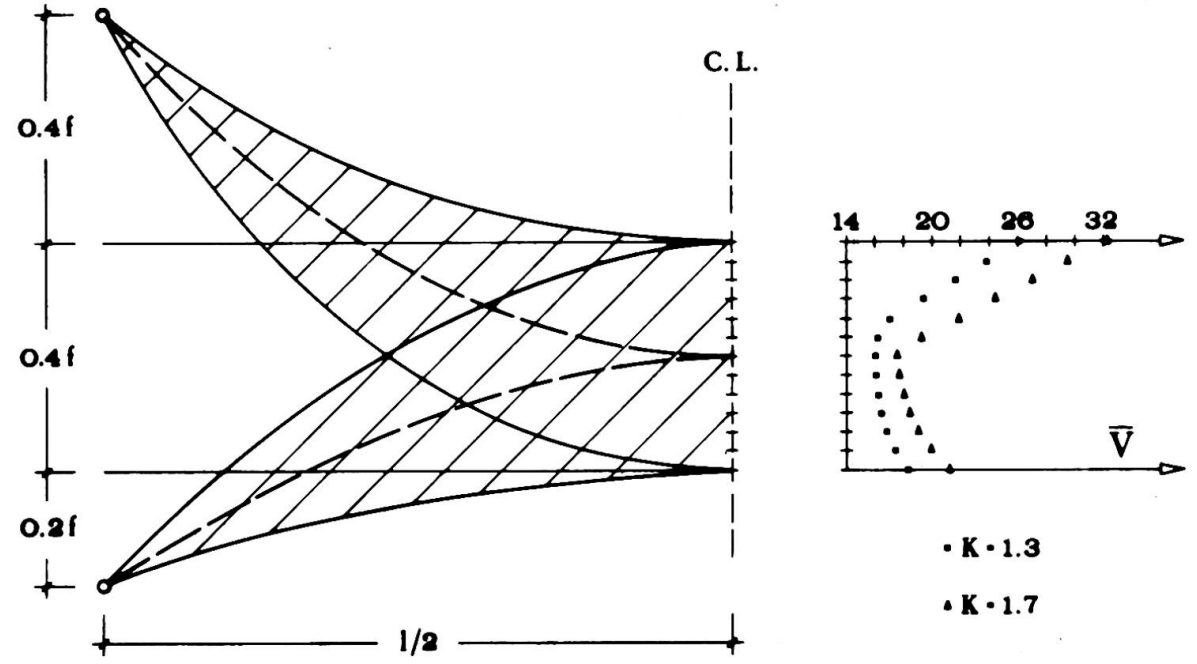

Fig. 2

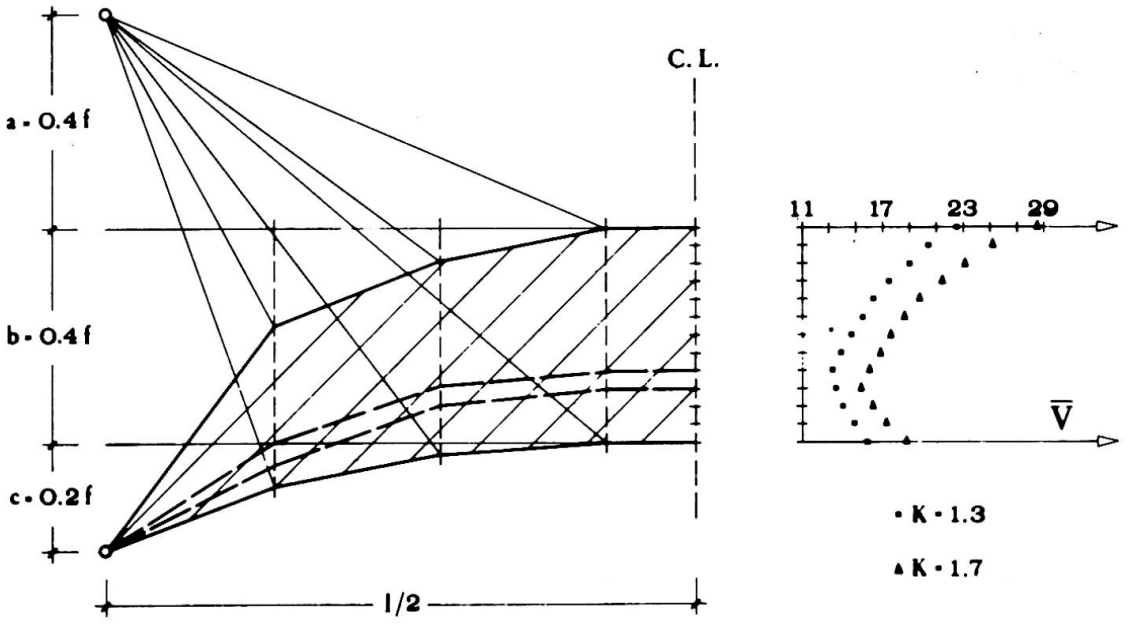

Fig. 3

The TSHS are studied with continuous approach: the relevant computer program (5) allows direct calculation of the areas of main cables and connecting roads once the geometry is fixed.

The PCSS are calculated by a discrete approach, specifically by displacement method: bar cross-sectional areas, when the geometry is fixed, are calculated by successive iterations (6) with a somewhat rapid convergence.

As an exemplification some results are graphically reported in Fig. 2 and in Fig. 3.

In either cases $f/l = 0.10, \bar{p} = |p_{max}| / |p_{min}| = 5.00$, $K = 1.3 \div 1.7$ (K is a number which quantifies the initial prestress) and $\bar{V} = (V\sigma_a)/(|p_{min}|l^2)$, where V is the volume of structure and σ_a is the allowable normal stress.

As it known, the fully stressed design of redundant structures coincides with the optimum design in peculiar situations only, which do not occur here.

The global optimum is determined by a process of successive iterations similar to Pedersen's (1). For each iteration, the problem of moving the design variables (cross-sectional areas and joint displacements) is worked out as a Linear Programming problem. Moreover, thanks to preliminary researches developed by fully stressed design, which gives solutions quite near to the global optimum, local minima can be avoided.

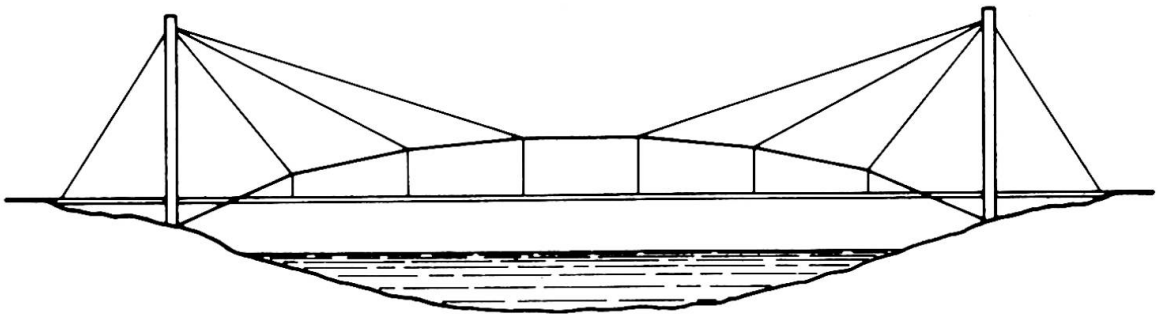

Fig. 4

The optimization process has emphasized the advantages of PCSS over TSHS when volume is the merit function. On the other hand, PCSS have a much better static and dynamic behaviour than TSHS, as it has been largely demonstrated (3), (4).

The extension of the proposed scheme [Fig. 1,b] to long span bridges [see Fig. 4] seems therefore reasonable. To this aim, it is necessary to conform the computational technique already formulated. This will be the purpose of future researches, where the effects of the moving loads and of the wind will be specifically considered.

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SUMMARY

The optimization of prestressed plane cable structures is presented and in particular the optimum design of a new kind of structure, whose good static and dynamic behaviour has already been reported on. Some conclusions and suggestions for future developments are given.

RESUME

On présente l'optimisation de structures planes de câbles prétendues et particulièrement le calcul optimal d'une nouvelle structure, dont une étude a déjà souligné le bon comportement statique et dynamique. Quelques conclusions et idées sont présentées pour de futurs développements.

ZUSAMMENFASSUNG

Es wird über die Optimierung von ebenen vorgespannten Seiltragwerken berichtet und besonders über die optimale Geometrie eines neuartigen Tragwerks; das gute statische und dynamische Verhalten dieses Tragwerks wurde schon früher hervorgehoben. Einige Bemerkungen über die zukünftigen Entwicklungsmöglichkeiten beschliessen diesen Beitrag.

Optimierung der Abmessungen vorgespannter Stahlvollwandträger

Optimization of Dimensions of Prestressed Steel Girders

Optimisation des dimensions de poutres métalliques précontraintes

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Stahlvollwandträger, die durch ein gerades hochfestes Zugband in der Ebene des Untergurtes vorgespannt sind, bilden das meistverwendete Grundelement vorgespannter Stahlkonstruktionen.

Bei der Ableitung rechnerischer Beziehungen für den optimalen Entwurf des durch gerades Zugband vorgespannten Vollwandträgers ist es notwendig, die Beziehungen unter einzelnen Querschnittscharakteristiken des Trägers zu kennen. Mit genügender Genauigkeit ist es möglich, alle Querschnittsgrössen in Abhängigkeit von der Trägerquerschnittsfläche F unter Anwendung folgender drei Parameter auszudrücken: Trägerasymmetrie $A = W_{x1}/W_{x2} = h_2/h_1$, Stegbeiwert $\varphi_{st} = F_{st}/F$, Stegschlankheit $\lambda = h/d$.

Bei der Ableitung der Beziehungen für den optimalen Entwurf des vorgespannten Vollwandträgers wird vom Festigkeitsgesichtspunkt ausgegangen. Die grösste Tragfähigkeit hat der Träger mit der optimalen Asymmetrie, in dem /bei verschiedenen Belastungsstadien/ möglichst viele Stellen ausgenützt sind, bei voll ausgenütztem Zugband - Bild 1.

In der Tabelle I. sind übersichtlich die Grundbeziehungen für die Berechnung der geometrischen Grössen und der Kraftgrössen des asymmetrischen Querschnittes I, beim optimalen Entwurf des vorgespannten Trägers angeführt. Die optimale Zugbandlänge ℓ_v folgt aus den Festigkeitsbedingungen des Trägers an den Stellen des Zugbandanfangs und Zugbandendes.

Als erste sind die Lösungsergebnisse des vorgespannten Trägers mit freier /nicht vorgeschriebener/ Untergurtfläche F_2 angeführt, d. h. die Fläche F_2 ist ausser Festigkeitsbedingung durch nichts beschränkt. Der Entwurf des vorgespannten Vollwandträgers wird beschleunigt, wenn die Bemessungsbeiwerte φ, φ_1, φ_{st}, φ_2, φ_v und ψ_v /Tabelle I./ und das Verhältnis ℓ_v/ℓ berechnet und vereinfacht werden in der Form eines Berechnungshilfsmittels für oft vorkommende Grundbelastungen. Einige Typen aus den Hilfsmitteln für den Entwurf sind auf den Bildern 2a und 2b angeführt. Teil des Hilfsmittels zur Bestimmung der Zugbandlänge ℓ_v ist am Bild 3.

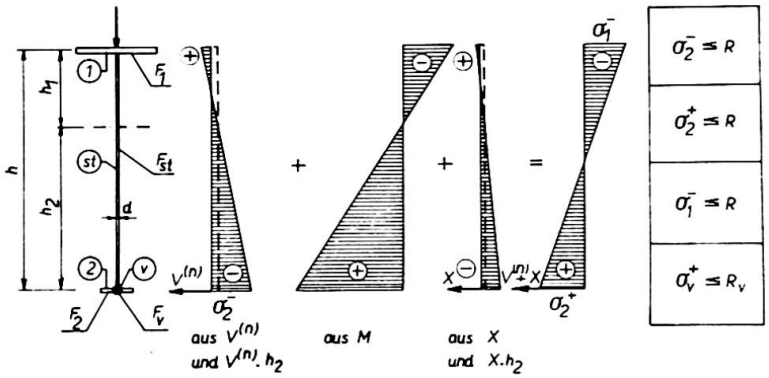

Bild 1. Spannungsbild bei einstufiger Vorspannung. $V^{(n)}$ - Normwert der Vorspannkraft; M - maximales Berechnungsbiegemoment aus der Belastung am Träger ohne Zugband; X - statisch unbestimmte Komponente der Zugbandkraft aus der Berechnungsbelastung; R - Berechnungsbeanspruchung /Index v ist für das Zugbandmaterial/

Tabelle I.

$F = \varphi \sqrt[3]{\frac{M^2}{R^2 \lambda}}$	$\varphi = \sqrt[3]{\frac{1}{C^2}}$,
	$C = \sqrt{\varphi_{st} \frac{3A - \varphi_{st}(A+1)}{3(A+1)}}$,
$F_1 = \varphi_1 F$	$\varphi_1 = \frac{A}{A+1} - \frac{\varphi_{st}}{2}$,
$F_{st} = F \varphi_{st}$	$\varphi_{st} = \frac{2[1 - \varphi_2(A+1)]}{A+1}$,
$F_2 = \varphi_2 F$	$\varphi_2 = \frac{1}{A+1} - \frac{\varphi_{st}}{2}$,
$F_v = F \frac{R}{R_v} \varphi_v$	$\varphi_v = D\left(1 - \frac{n_{vd}}{n_{vh}}\right) \cdot \frac{A-1}{A+1}$,
	$D = \frac{6A - \varphi_{st}(A+1)^2}{(A+1)[6A - \varphi_{st}(A+1)]}$,
$V^{(n)} = FR\psi_v$	$\psi_v = \frac{D}{n_{vh}}$

Auf den Bildern 2a und 2b ist der Bereich mit $F_2 < 0{,}04.F$ bei den betreffenden Bemessungsbeiwerten gestrichelt angezeigt; es hat keinen Zweck $F_2 < 0{,}04.F$ zu entwerfen. In diesem Falle wird die Grösse der Trägeruntergurtfläche F_2 durch die Beziehung

$$F_2 = \varphi_2 F = \left/ \frac{1}{A+1} - \frac{\varphi_{st}}{2} \right/ F$$

gebunden /vorgeschrieben/.

Die Berechnung der notwendigen Bemessungsbeiwerte für diesen zweiten Fall, d. i. vorgespannter Träger mit vorgeschriebener /gebundener/ Untergurtfläche F_2, ist ähnlich wie im ersten Fall, aber es muss auch die oben angeführte Bedingung erfüllt sein.

Wie man aus der Beziehung für φ_{st} /Tabelle I./ sieht, befindet sich in ihr nur eine Unbekannte und zwar Querschnittsasymmetrie A, da φ_2 gewählt wird; es bleibt nur der Beiwert A zu berechnen.

Durch Vergleich der Beziehungen für die Berechnung der Kraft X im Zugband, die aus den Festigkeits- und Verformungsbedingungen bestimmt sind, erhalten wir schliesslich die Beziehung

$$\frac{\varrho}{1 + (A-1)\dfrac{6A - 2[1 - \varphi_2(A+1)]}{6A - (A+1)2[1 - \varphi_2(A+1)]} - \dfrac{n_{vd}}{n_{vh}}} - \alpha \frac{A\{6A - 4[1 - \varphi_2(A+1)]\}}{\{6A - 2[1 - \varphi_2(A+1)](A+1)\} \cdot \left\{(A-1)\dfrac{6A - 2(1 - \varphi_2(A+1)]}{6A - 2[1 - \varphi_2(A+1)](A+1)} - \dfrac{n_{vd}}{n_{vh}}\right\}} = 0$$

aus der die optimale Asymmetrie A in Abhängigkeit vom den gewählten Parameter E, E_v, R, R_v, φ_2, n_{vd}/n_{vh} und von der Belastungsart bestimmt wird; n_v ist der Koeffizient der Vorspanngenauigkeit /$n_{vd} \leq 1{,}0$ - im allgemeinen 0,9, $n_{vh} \geq 1{,}0$ - im allgemeinen 1,1/, $\varrho = ER_v/E_vR$. Nach Bestimmung des Wertes A werden mit seiner Hilfe

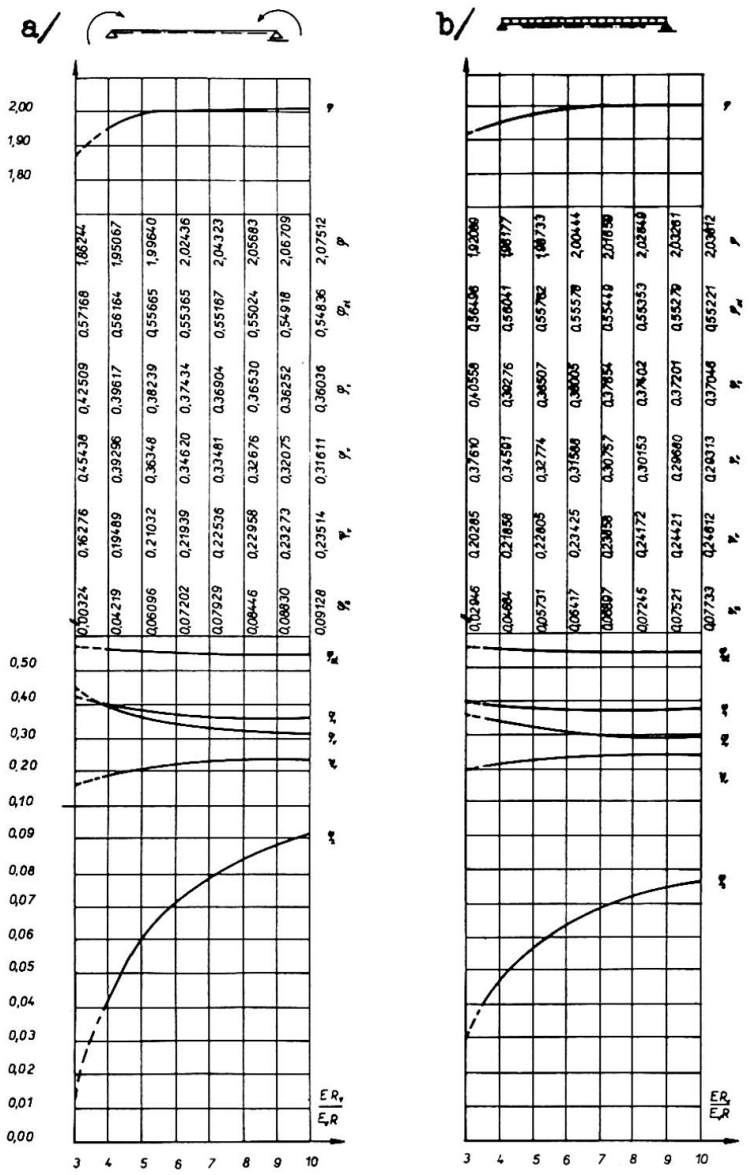

Bild 2. Bemessungsbeiwerte für den Entwurf des vorgespannten Vollwandträgers:
a/ $R_1 = R_2 = R$, $n_{vh} = 1{,}1$ und $n_{vd} = 0{,}9$;
b/ $R_1 = R_2 = R$ und $n_{vh} = n_{vd} = 1{,}0$

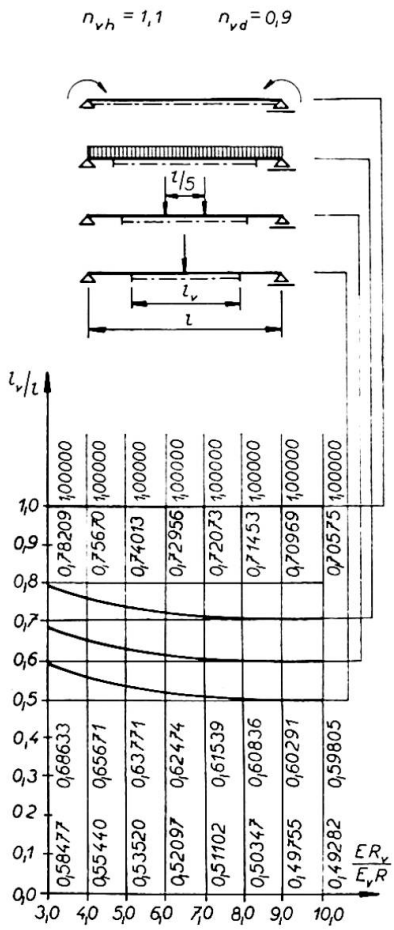

Bild 3. Verhältnislängen l_v/l ; die Zahlenwerte entsprechen den zugehörigen Belastungen in der Reihenfolge wie die Belastung angeführt ist /$R_1 = R_2 = R$/

alle weiteren Beiwerte berechnet, die für den optimalen Entwurf des vorgespannten Vollwandträgers gebraucht werden, und zwar φ, φ_1, φ_{st}, φ_v, ψ_v und l_v/l .

Als Beispiel ist aus der grossen Zahl der Hilfsmittel nur eines angeführt /Bild 4/, mit der grafischen Darstellung und den Zahlenwerten der Bemessungsbeiwerte, die für den optimalen Entwurf des Stahlträgers, der mit geradem Zugband vorgespannt ist, gebraucht werden; der Wert φ_2 ist gewählt. Aus der grossen Zahl tabellarischer Hilfsmittel ist nur eine in der Tabelle II. angeführt.

Der Berechnungsvorgang beim Entwurf ist derselbe wie bei den Trägern mit freier Untergurtfläche; unterschiedlich ist dabei, dass im zweiten Fall φ_2 beim Entwurf gewählt wird.

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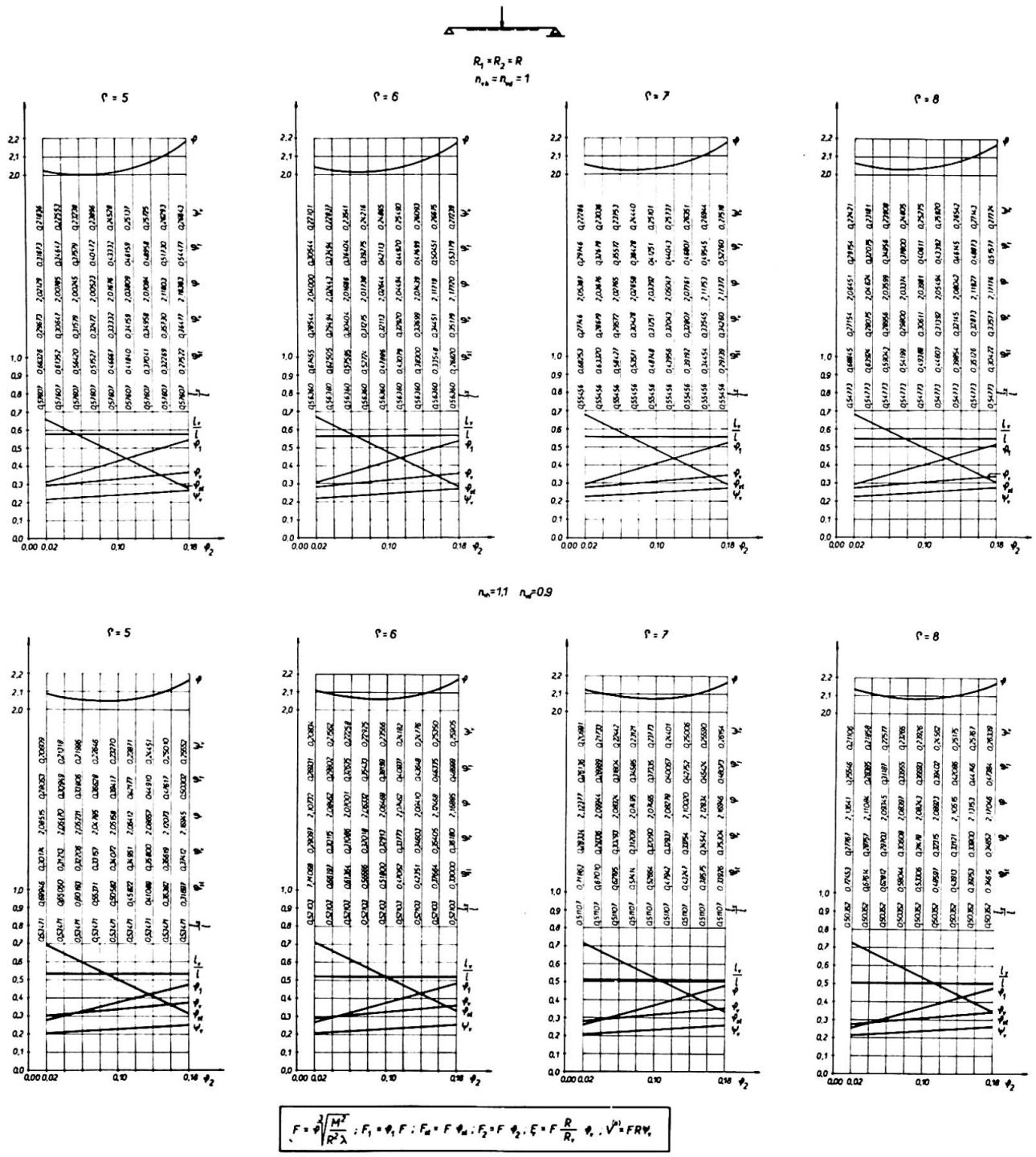

Bild 4. Bemessungsbeiwerte und Beziehungen für den Entwurf des vorgespannten Stahlträgers; Wert φ_2 ist gewählt

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Tabelle II. Hilfs- und Ergebnisbeiwerte für den Entwurf des vorgespannten Stahlträgers; Wert φ_2 ist gewählt. $R_1 = R_2 = R$ und $n_{vh} = n_{vd} = 1{,}0$

ϱ	φ_2	A	α	φ_{st}	φ_v	C	D	φ	φ_1	μ_l	ψ_v
	0,02	2,97355	1,00000	0,46332	0,49667	0,40425	0,16555	1,82907	0,51667	1,00000	0,16555
	0,04	3,07985	1,00000	0,41021	0,50978	0,39591	0,16992	1,85466	0,54978	1,00000	0,16992
	0,06	3,18765	1,00000	0,35759	0,52240	0,38391	0,17413	1,89311	0,58240	1,00000	0,17413
	0,08	3,29714	1,00000	0,30542	0,53457	0,36777	0,17819	1,94808	0,61457	1,00000	0,17819
3	0,10	3,40850	1,00000	0,25366	0,54633	0,34682	0,18211	2,02579	0,64633	1,00000	0,18211
	0,12	3,52193	1,00000	0,20228	0,55771	0,31997	0,18590	2,13758	0,67771	1,00000	0,18590
	0,14	3,63762	1,00000	0,15125	0,56874	0,28544	0,18958	2,30666	0,70874	1,00000	0,18958
	0,16	3,75573	1,00000	0,10054	0,57945	0,23978	0,19315	2,59090	0,73945	1,00000	0,19315
	0,18	3,87646	1,00000	0,05013	0,58986	0,17424	0,19662	3,20545	0,76986	1,00000	0,19662
	0,02	2,27839	1,00000	0,57005	0,38994	0,38125	0,19497	1,90192	0,40994	1,00000	0,19497
	0,04	2,34116	1,00000	0,51859	0,40140	0,38011	0,20070	1,90571	0,44140	1,00000	0,20070
	0,06	2,40367	1,00000	0,46760	0,41239	0,37632	0,20619	1,91848	0,47239	1,00000	0,20619
	0,08	2,46602	1,00000	0,41702	0,42297	0,36969	0,21148	1,94136	0,50297	1,00000	0,21148
4	0,10	2,52830	1,00000	0,36684	0,43315	0,35995	0,21657	1,97622	0,53315	1,00000	0,21657
	0,12	2,59060	1,00000	0,31700	0,44299	0,34673	0,22149	2,02614	0,56299	1,00000	0,22149
	0,14	2,65297	1,00000	0,26749	0,45250	0,32950	0,22625	2,09617	0,59250	1,00000	0,22625
	0,16	2,71548	1,00000	0,21828	0,46171	0,30746	0,23085	2,19515	0,62171	1,00000	0,23085
	0,18	2,77820	1,00000	0,16935	0,47064	0,27937	0,23532	2,33997	0,65064	1,00000	0,23532
	0,02	2,05107	1,00000	0,61550	0,34449	0,36644	0,20869	1,95282	0,36449	1,00000	0,20669
	0,04	2,10151	1,00000	0,56484	0,35515	0,36773	0,21309	1,94824	0,39515	1,00000	0,21309
	0,06	2,15142	1,00000	0,51463	0,36536	0,36668	0,21921	1,95197	0,42536	1,00000	0,21921
	0,08	2,20089	1,00000	0,46482	0,37517	0,36314	0,22510	1,96461	0,45517	1,00000	0,22510
5	0,10	2,25000	1,00000	0,41538	0,38461	0,36695	0,23076	1,98727	0,48461	1,00000	0,23076
	0,12	2,29880	1,00000	0,36627	0,39372	0,34785	0,23623	2,02178	0,51372	1,00000	0,23623
	0,14	2,34737	1,00000	0,31748	0,40251	0,33549	0,24150	2,07112	0,54251	1,00000	0,24150
	0,16	2,39574	1,00000	0,26897	0,41102	0,31939	0,24861	2,14015	0,57102	1,00000	0,24661
	0,18	2,44396	1,00000	0,22072	0,41927	0,29883	0,25156	2,23725	0,59927	1,00000	0,25156
	0,02	1,93850	1,00000	0,64061	0,31938	0,35709	0,21292	1,98675	0,33938	1,00000	0,21292
	0,04	1,98316	1,00000	0,59042	0,32957	0,35958	0,21971	1,97755	0,36957	1,00000	0,21971
	0,06	2,02719	1,00000	0,54067	0,33932	0,35988	0,22621	1,97647	0,39932	1,00000	0,22621
	0,08	2,07069	1,00000	0,49131	0,34868	0,35787	0,23245	1,98385	0,42868	1,00000	0,23245
6	0,10	2,11374	1,00000	0,44231	0,35768	0,35341	0,23845	2,00050	0,45768	1,00000	0,23845
	0,12	2,15638	1,00000	0,39363	0,36636	0,34630	0,24424	2,02779	0,48636	1,00000	0,24424
	0,14	2,19868	1,00000	0,34525	0,37474	0,33626	0,24982	2,06796	0,51474	1,00000	0,24982
	0,16	2,24067	1,00000	0,29715	0,38284	0,32291	0,25523	2,12459	0,54284	1,00000	0,25532
	0,18	2,28241	1,00000	0,24930	0,39069	0,30569	0,26046	2,20363	0,57069	1,00000	0,26046
	0,02	1,87138	1,00000	0,65652	0,30347	0,35075	0,21876	2,01061	0,32347	1,00000	0,21676
	0,04	1,91266	1,00000	0,60665	0,31334	0,35396	0,22381	1,99844	0,35334	1,00000	0,22381
	0,06	1,95330	1,00000	0,55720	0,32279	0,35506	0,23056	1,99432	0,38279	1,00000	0,23056
	0,08	1,99337	1,00000	0,50814	0,33185	0,35395	0,23704	1,99846	0,41185	1,00000	0,23704
7	0,10	2,03293	1,00000	0,45942	0,34057	0,35052	0,24326	2,01149	0,44057	1,00000	0,24326
	0,12	2,07206	1,00000	0,41102	0,34897	0,34458	0,24926	2,03455	0,46897	1,00000	0,24926
	0,14	2,11079	1,00000	0,36292	0,35707	0,33589	0,25505	2,06949	0,49707	1,00000	0,25505
	0,16	2,14917	1,00000	0,31508	0,36491	0,32412	0,26065	2,11929	0,52491	1,00000	0,26065
	0,18	2,18724	1,00000	0,26750	0,37249	0,30881	0,26606	2,18877	0,55249	1,00000	0,26606
	0,02	1,82682	1,00000	0,66750	0,29249	0,34620	0,21936	2,02820	0,31249	1,00000	0,21936
	0,04	1,86591	1,00000	0,61785	0,30214	0,34988	0,22660	2,01397	0,34214	1,00000	0,22660
	0,06	1,90433	1,00000	0,56862	0,31137	0,35150	0,23352	2,00775	0,37137	1,00000	0,23352
	0,08	1,94216	1,00000	0,51977	0,32022	0,35100	0,24017	2,00968	0,40022	1,00000	0,24017
8	0,10	1,97948	1,00000	0,47125	0,32874	0,34824	0,24655	2,02028	0,42874	1,00000	0,24655
	0,12	2,01632	1,00000	0,42305	0,33694	0,34306	0,25270	2,04053	0,45694	1,00000	0,25270
	0,14	2,05274	1,00000	0,37514	0,34485	0,33526	0,25863	2,07208	0,48485	1,00000	0,25863
	0,16	2,08880	1,00000	0,32750	0,35249	0,32452	0,26437	2,11754	0,51249	1,00000	0,26437
	0,18	2,12451	1,00000	0,28009	0,35990	0,31044	0,26992	2,18110	0,53990	1,00000	0,26992
	0,02	1,79509	1,00000	0,67553	0,28446	0,34277	0,22124	2 04169	0,30446	1,00000	0,22124
	0,04	1,83262	1,00000	0,62605	0,29394	0,34678	0,22862	2 02592	0,33394	1,00000	0,22862
	0,06	1,86950	1,00000	0,57698	0,30301	0,34879	0,23567	2 01816	0,36301	1,00000	0,23567
	0,08	1,90576	1,00000	0,52828	0,31171	0,34870	0,24244	2 01849	0,39171	1,00000	0,24244
9	0,10	1,94149	1,00000	0,47992	0,32007	0,34642	0,24894	2 02733	0,42007	1,00000	0,24894
	0,12	1,97674	1,00000	0,43187	0,32812	0,34179	0,25520	2 04559	0,44812	1,00000	0,25520
	0,14	2,01156	1,00000	0,38410	0,33589	0,33461	0,26124	2 07476	0,47589	1,00000	0,26124
	0,16	2,04599	1,00000	0,33660	0,34339	0,32460	0,26708	2 11720	0,50339	1,00000	0,26708
	0,18	2,08006	1,00000	0,28933	0,35066	0,31138	0,27273	2 17672	0,53066	1,00000	0,27273
	0,02	1,77135	1,00000	0,68166	0,27833	0,34011	0,22266	2 05234	0,29833	1,00000	0,22266
	0,04	1,80773	1,00000	0,63231	0,28768	0,34436	0,23014	2 03540	0,32768	1,00000	0,23014
	0,06	1,84345	1,00000	0,58337	0,29662	0,34665	0,23730	2 02646	0,35662	1,00000	0,23730
	0,08	1,87855	1,00000	0,53479	0,30520	0,34688	0,24416	2 02556	0,38520	1,00000	0,24416
10	0,10	1,91311	1,00000	0,48655	0,31344	0,34495	0,25075	2 03308	0,41344	1,00000	0,25075
	0,12	1,94718	1,00000	0,43861	0,32138	0,34073	0,25710	2 04984	0,44138	1,00000	0,25710
	0,14	1,98082	1,00000	0,39095	0,32904	0,33401	0,26323	2 07723	0,46904	1,00000	0,26323
	0,16	2,01405	1,00000	0,34355	0,33644	0,32454	0,26915	2 11746	0,49644	1,00000	0,26915
	0,18	2,04692	1,00000	0,29640	0,34359	0,31195	0,27487	2 17405	0,52359	1,00000	0,27487

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ZUSAMMENFASSUNG

Der Aufsatz befasst sich mit der Optimierung der Abmessungen vorgespannter Stahlvollwandträger. Angeführt werden Berechnungshilfsmittel für den Entwurf, wobei die Trägeruntergurtfläche F_2 im ersten Fall frei und im zweiten Fall gebunden ist.

SUMMARY

This article deals with the optimization of dimensions of prestressed steel girders. Calculation means are given for the design, whereas in the first case the area of the lower flange F_2 is assumed to be free and in the second case it is assumed to be given.

RESUME

L'article traite de l'optimisation des dimensions des poutres métalliques précontraintes. Des tables auxiliaires de calcul sont présentées, avec la section de la semelle inférieure F_2 libre en premier cas et liée en deuxième cas.

Optimization of the Prestressing in the Cables of a Cable-Stayed Bridge

Optimisation de la précontrainte dans les câbles d'un pont à haubans

Optimierung der Vorspannung in den Kabeln einer Schrägseilbrücke

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1. Reasons for prestressing

It is well known that the cables of a cable-stayed bridge have to be prestressed in order to utilize the high strength of the cables and to reduce the bending moments in the main girder.

The degree of prestressing required depends on the structural system of the bridge, the relative stiffness of the cables and the girder, and the ratio of live load to dead load. Finding the most economic relation between the longitudinal stiffness of the cables and the bending stiffness of the main girder is a complicated optimization problem in itself.

Here, only a short description is given of how prestress in the cables was optimized for a given structural system. The aim was to get a uniform distribution of moments in the main girder without overloading the cables.

2. The Deggenau cable-stayed bridge

The bridge in question carries a motorway across the Danube in Eastern Bavaria and has two spans of unequal length, i.e. 290 m and 145 m (Fig. 1). It has one central tower and the large opening is back-stayed by three cables to the small opening and mainly to the eastern abutment. The main girder has a two cell box-section of 14 x 4.5 m with cantilevering deck supported by struts.

The cables are composed of spiral strands arranged in three layers in a rectangular pattern (Fig. 2), for more details about this bridge see [1].

3. Optimality conditions

In static analysis the bridge was treated as a space frame in linear elastic theory with the aid of a standard computer program.

The loading case "end displacement" of a certain cable then delivers a complete set of influence coefficients reflecting the effect of prestressing. Thus, from the loading cases "end displacement" of all the cables one can easily obtain the coefficients for the value of any internal force or moment due to prestressing:

$$S_i = \sum_k a_{ik} X_k$$

with S_i = internal force or moment
a_{ik} = influence coefficient
X_k = prestressing (end displacement of cable k)

The optimality criteria to be considered were the following (see also Fig. 3):

a) The tensions in the cables were not to exceed the maximum allowable values and should not become negative.

b) The moments in the main girder were not to exceed the strength of the girder required for cantilever erection.

c) The moments at the tower base were to be kept to a minimum.

With 6 different cables, 6 positive and 5 negative limit moments for the main girder, and two limit moments for the tower base this gives a set of 25 inequality conditions for the 6 unknown cable end displacements.

4. Practical solution

Assuming that the system would behave inherently reasonable, the number of inequalities was reduced to 19 by dropping the condition that cable tensions should not become negative.

A further reduction was possible due to the fact that positive bending moments in the girder between cable points were not critical.

It was then tried to solve the remaining set of 13 inequalities by means of a standard computer program for linear programming. As it turned out this program failed to give a reasonable solution because, being geared to economic problems only, it could not handle negative values on the right hand side. Merely changing signs all through the equations concerned did not help.

Since there was neither time nor a specialist available to overcome these difficulties, the number of variables and restrictions was further reduced by engineering judgement. The remaining set of 10 inequalities with 5 unknowns was handled as overdetermined system of linear equations by a computer program producing a least squares solution. This approach gave surprisingly satisfactory results for all the values that were to be optimized.

References

[1] Feder, D.: Donaubrücke Deggenau, Strasse Brücke Tunnel 5 (1975), S. 114/17

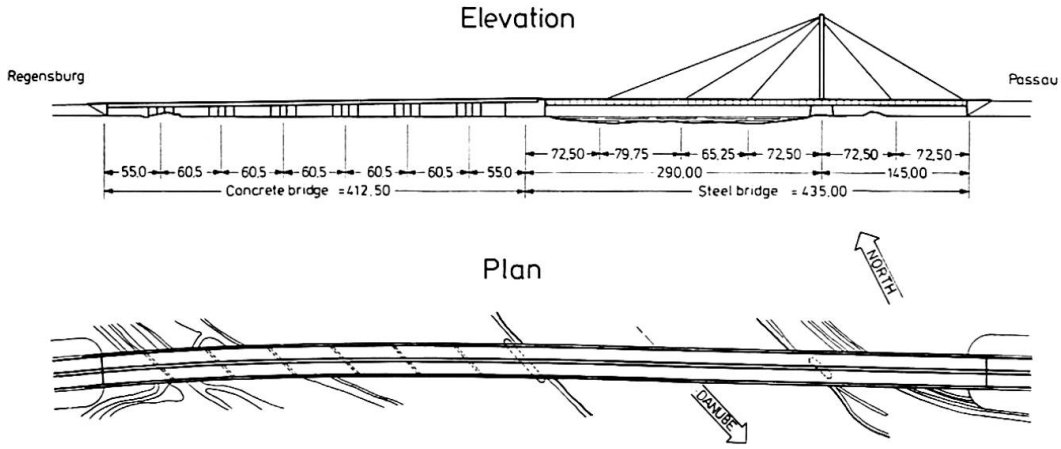

Fig. 1: Deggenau cable-stayed bridge

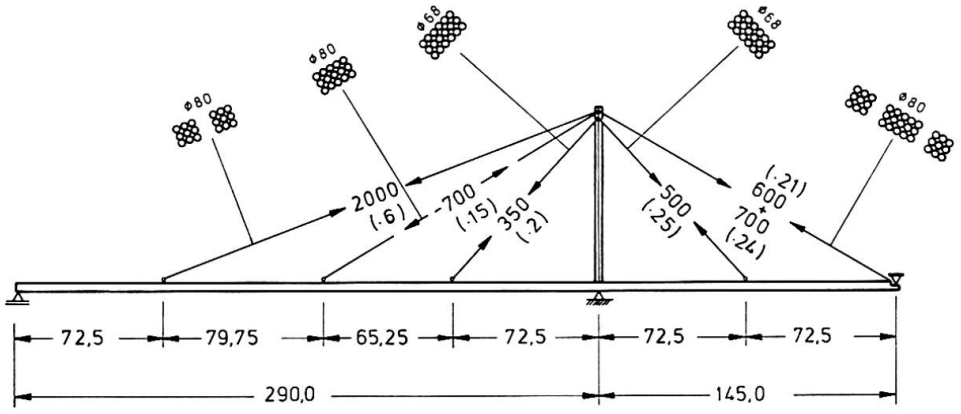

Fig. 2: Cable sections and prestressing:
prestress [kN], (cable end displacements [m])

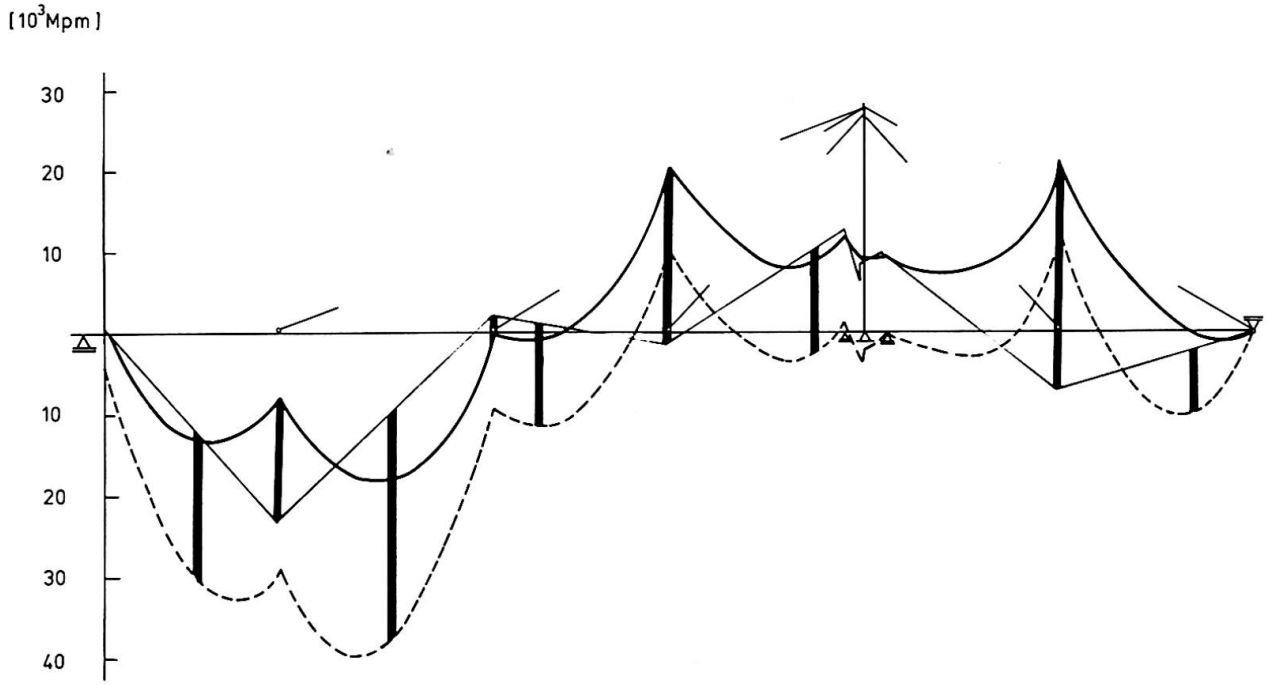

Fig. 3: Optimality criteria and moment distribution
in main girder

SUMMARY

The paper deals with an optimization that has been used in the design of a cable-stayed bridge. The optimality criteria are briefly discussed, difficulties encountered are described, and the engineering approach applied for the solution is outlined.

RESUME

L'article traite de l'optimisation retenue pour le calcul d'un pont à haubans. Les critères d'optimisation sont discutés, les difficultées rencontrées sont décrites, et la méthode d'ingénieur employée est présentée.

ZUSAMMENFASSUNG

Der Beitrag befasst sich mit einer Optimierung, die bei der Berechnung einer Schrägseilbrücke benutzt wurde. Es werden kurz die Optimalitätskriterien diskutiert, aufgetretene Schwierigkeiten werden beschrieben, und das für die Lösung angewendete ingenieurmässige Vorgehen wird umrissen.

Optimierung elastischer, ebener Rahmentragwerke

Optimization of Elastic Plane Frames

Optimisation des cadres plans élastiques

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1. Einführung

Die Optimierung der Tragwerke wurde im Einführungsbericht /1/ in 4 Kategorien eingeteilt. Der vorliegende Bericht beschreibt ein Verfahren, das der 3. Kategorie angehört und gleichzeitig die Aufgabestellung der 4. Kategorie weitgehend erfüllt. Ebene Rahmentragwerke mit linear-elastischem Last-Verformungszusammenhang sollen bei gegebenen Geometrie-, Lagerbedingungen und Belastungszuständen so dimensioniert werden, daß ein Optimalitätskriterium für das gesamte Tragwerk erfüllt und die einzelnen Elemente des diskretisierten Tragwerkes für sich optimal gestaltet sind.

2. Zielfunktion

Mit den Querschnittsflächen A_i als Variablen ist die Zielfunktion für minimales Gewicht oder minimale Kosten zu formulieren. Voraussetzung ist eine stetige und stetig differenzierbare Funktion.

$$f_{(A_i)} \Rightarrow \min \qquad i = 1.....n \qquad (1)$$

3. Optimaler Einzelquerschnitt

Der optimale Einzelquerschnitt ist durch die Erzielung des erforderlichen Biegewiderstandes bei minimaler Querschnittsfläche gekennzeichnet. Die Zusammenhänge zwischen Trägheitsmoment I, Widerstandsmoment W, Bauhöhe h und optimaler Querschnittsfläche sind nach Festlegung der Querschnittsform durch Differentialrechnung zu ermitteln. Unter Verwendung üblicher Näherungen ergibt sich beispielsweise am doppelt-symmetrischen I-Profil, dessen Stegverhältnis $\lambda=h/t$ den Erfordernissen der Stegbeulung entsprechend vorgewählt wird

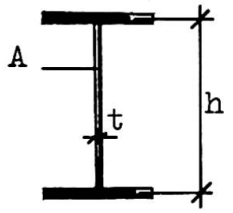

Abb.1. Bezeichnungen am I-Querschnitt

$$\left.\begin{aligned} I &= \frac{\lambda}{12} \cdot A^2 \\ W &= \left(\sqrt[3]{\frac{16}{3\lambda}} + \sqrt[3]{\frac{2}{3\lambda}} \right)^{-3/2} \cdot A^{3/2} \\ h &= \sqrt{\frac{\lambda}{2}} \cdot A^{1/2} \end{aligned}\right\} \quad (2)$$

Bei einfach-symmetrischen Profilen sind die Gurtflächen als getrennte Variablen einzuführen.

4. Restriktionssystem

Die nicht-linearen Restriktionen für die Variablen A_i sind durch die Forderung gegeben, daß äußere Schnittgrößen an keiner Stelle des Tragwerkes die inneren Widerstände übersteigen dürfen.

$$\left.\begin{aligned} |M^{ä}| &\leqslant |M^{i}| \\ |N^{ä}| &\leqslant |N^{i}| \end{aligned}\right\} \quad (3)$$

Der Einfluß der Querkräfte bleibt unberücksichtigt.
Als Vorsorge gegen Stabilitätsversagen des Systems werden die äußeren Schnittgrößen unter Berücksichtigung von Vorverformungen und ungewollten Lastausmitten im Traglastzustand nach Theorie II. Ordnung errechnet.

Bei Verwendung der verallgemeinerten Deformationsmethode wird die globale Steifigkeitsmatrix $[S]$ durch Summation aus den ins globale System rotierten Stabsteifigkeitsmatrizen $[L]$ gewonnen.

$$[L] = [L^{I}] + [L^{II}] \quad (4)$$

Die Matrix I. Ordnung enthält die Variablen A_i, die Matrix II. Ordnung die unbekannten Stablängskräfte N. In der Iterationsrechnung dürfen - mit Rücksicht auf die geringe Änderung der Normalkräfte bei Variation der Steifigkeiten - die Längskräfte des vorhergehenden Iterationsschrittes als Konstantwerte verwendet werden.

Die Knotenverformungen $\{v\}$ unter der Belastung $\{P\}$ sind durch

$$\{v\} = [S]^{-1} \cdot \{P\} \quad (5)$$

gegeben. Die Änderung einer Verformung v_k kann im Bereich kleiner Steifigkeitsänderungen genügend genau durch

$$\Delta v_k = \text{grad } v_k \; \{\Delta A\} \quad (6)$$

angenähert werden, wobei grad v_k aus (5) numerisch ermittelt wird. Hierbei kann vorteilhaft von der Näherung

$$[S + \Delta S_i]^{-1} \doteq ([I] - [S]^{-1} [\Delta S_i]) . [S]^{-1} \quad (7)$$

Gebrauch gemacht werden /2/, in der $[I]$ die Einheitsmatrix und $[\Delta S_i]$ die Änderung der globalen Steifigkeitsmatrix durch Änderung des Querschnittes A_i bedeuten. Die Matrix $[S]$ muß in jedem Iterationsschritt nur einmal invertiert werden.

Sind $\{w\}$ die aus $\{v\}$ durch Zuordnung und Rotation ins lokale Koordinatensystem gewonnen Stabendverformungen, so gilt für die Schnittgrößen

$$\{M^{ä}\} = [L] \; \{w\} \quad (8)$$

und bei Variation der Flächen A_i

$$\{M^{ä}\} = [L+\Delta L] \; \{w+\Delta w\} \leqslant \{M^{i}\} \quad (9)$$

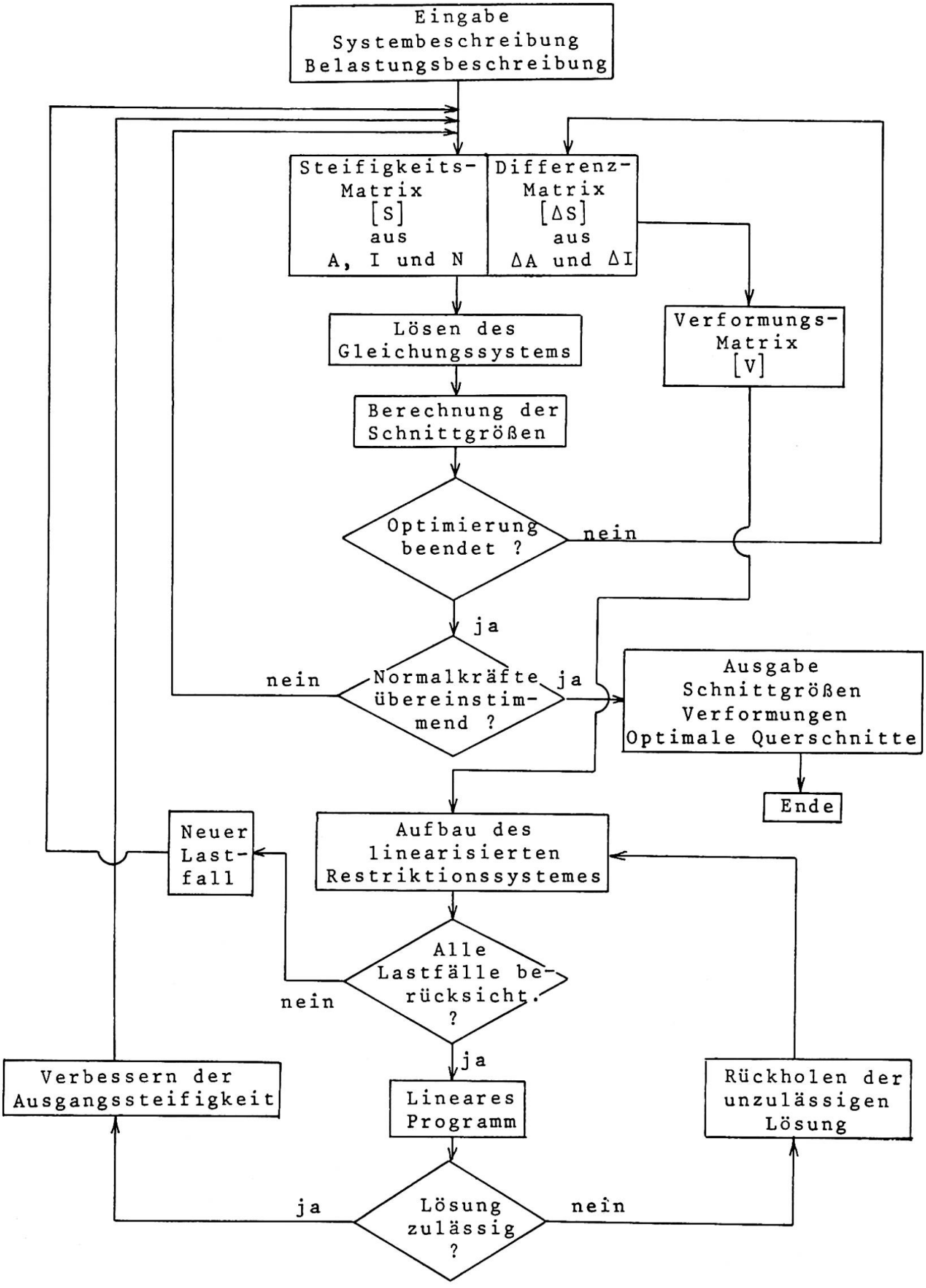

Abb. 2. Schematisches Flußdiagramm

Gleichung (9) an allen Stabenden und für alle Lastfälle formuliert liefert das Restriktionssystem für die Variablen A_i.

5. Mathematische Optimierung

Die Lösung der nicht-linearen Optimierungsaufgabe erfolgt mittels der "Methode der approximierenden Optimierung" /3/, einer Variante der Sequentiellen Linearen Programmierung. Zielfunktion und Restriktionen werden in einem zulässigen Startpunkt $\{A_o\}$ linearisiert und mittels Simplex-Algorithmus die Lösung des Linearen Programmes bestimmt. Der Lösungspunkt $\{\bar{A}\}$, der wegen des Linearisierungsfehlers außerhalb des zulässigen Raumes liegt, wird mit der Vektoraddition

$$\{A^*\} = \frac{1}{2^t} \cdot \{\bar{A}\} + \frac{2^t-1}{2^t} \cdot \{A_o\} \qquad (10)$$

rückgeführt. Die Potenz t wird beginnend mit t=0 solange um 1 erhöht, bis $\{A^*\}$ in den zulässigen Grundbereich zu liegen kommt.

Mit den verbesserten Werten für die Querschnittsflächen werden verbesserte Verformungsgradienten errechnet und ein neuerlicher Iterationszyklus durchlaufen.

Der Berechnungsablauf ist an einem schematischen Flußdiagramm erläutert. (Abb.2)

Das Konvergenzverhalten wurde an verschiedenen kleineren Beispielen untersucht. Die Verbesserung des Wertes der Zielfunktion nach 5 Iterationsschritten lag zumeist unter 1 %.

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ZUSAMMENFASSUNG

Der Bericht behandelt die Optimierung elastischer, ebener Rahmentragwerke. Die Lösung des nicht-linearen Programmes erfolgt mit der "Methode der approximierenden Optimierung", einer Variante der sequentiellen linearen Programmierung.

SUMMARY

The paper examines the optimization of elastic plane frames. The solution of the non-linear programming is obtained by the method "Methode der approximierenden Optimierung", a kind of sequential linear programming.

RESUME

L'article traite de l'optimisation des cadres plans élastiques. La solution pour la programmation non-linéaire est obtenue à l'aide de la "Methode der approximierenden Optimierung", une variante de la programmation linéaire séquentielle.

An Optimality Criterion Method for Composite Bridge Deck Design

Une méthode basée sur le critère de l'optimalité pour le calcul du tablier composite d'un pont

Eine Optimierungsmethode für die Berechnung von Verbunddecken im Brückenbau

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Gellatly and Dupree (1) have discussed some of the important limitations which arise from application of mathematical programming techniques to structural problems. The need amongst others for the evaluation of derivatives of objective functions and constraints in most mathematical programming methods often leads to the expenditure of large computer time for the solution of realistic size structures, whose accurate description depend on large numbers of design variables. Whilst recognising that some new developments are directed towards ameliorating some of these problems, there is yet no indications that all of them have been solved.

On the other hand the broad group of methods, classed as optimality criterion approach, employ completely different techniques from those of mathematical programming and are thereby free from the weaknesses of the latter, although they have their peculiar shortcomings. In this discussion an example of the application of the method to the optimum cost design of composite bridge deck is presented.

In developing an optimality criterion approach for this problem, it is considered necessary to introduce the following simplifying assumptions:

1. The width of the bridge deck is fixed and the configurations of the deck are as shown in Figure 1.

2. For the steel girders, only universal beam sections with tabulated section properties from manufacturers are used.

3. The design of shear connectors is not considered although it is assumed that adequate shear connectors are provided between the slab and the beam to make it possible to use transformed section theory.

4. Shored construction is assumed to reduce the number of load cases to be considered.

The characteristics of the objective function are studied (2) through the aid of a computer program written to design the deck to the requirements of the British Code for Composite Construction (CP 117 1967 Part 2) and for Design of Steel Girder Bridges (BS 153 1966 Part 3 & 4).

Amongst the characteristics studied are:

1. The variation of deck cost with depth of slab for different cost of concrete, μ for a given girder. A typical example Figure 2 shows that the cost of concrete has very little influence on the cost of the deck.

2. The variation of deck cost with depth of slab for different cost of steel, λ. Figure 3 shows that the cost of girder has a predominant effect on the cost of the deck.

3. The variation of the deck cost with depth of slab for girders of the same serial size but of different weights. Figure 4 for four 914mm x 305mm girders shows that if a feasible design region exists with the possibility of choice of girders, the optimum design is obtained for the girder with the lowest value of weight per unit length.

These points dictate the mode of procedure for the optimization process. It is considered reasonable from the relatively minor contribution of the slab cost to choose, as a first approach, a slab depth based only on satisfaction of strength and deflection constraints. The table of girder section properties is ordered with respect to the weight per unit length to create a logical direction of search. A direct search method (3) with variable travel steps, controlled by a sensitivity device, is used to determine the optimum girder.

One of the main points to be emphasised here is the fact that the assumptions stated above, though restrictive, are necessary for application of the concept of optimality criteria. Secondly the knowledge of the characteristics of the problem is the principal factor responsible for the easy development of the simple direct search scheme. Lastly the problem falls into the first class of hierarchy discussed by Gellatly and Dupree (1) and further illustrates the point that optimality criterion methods are very efficient at dealing with only one or two of the hierarchy at a time.

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Optimum Design of Composite Bridge Deck Using Available Universal Beams.
To be published.

APPENDIX 1 - NOTATION

C_c Cost of concrete per cubic metre.

C_r Cost of reinforcement per cubic metre.

C_g Cost of girder per 100 kilogram.

$\lambda = C_g/C_c$

$\mu = C_r/C_c$

SUMMARY

An example of the application of an optimality criterion method to the design of composite bridge deck for minimum cost is presented. Simplifying assumptions which facilitated the application of the method are discussed. It is argued that a study of the characteristics of the problem contributed immensely to the development of a simple direct search scheme for the optimal solution.

RESUME

On donne un exemple de l'application d'une méthode basée sur le critère de l'optimalité pour le calcul du tablier composite d'un pont,pour des frais minima. Puis des hypothèses simplificatrices qui ont facilité l'application de cette méthode sont considérées. Il est montré qu'une étude des caractéristiques du problème a contribué énormément au développement de combinaisons simples et directes pour la recherche d'une solution optimale.

ZUSAMMENFASSUNG

Ein Beispiel für die Anwendung einer Optimierungsmethode zur Berechnung von Verbunddecken mit minimalen Kosten im Brückenbau wird angegeben. Vereinfachende Voraussetzungen werden diskutiert, welche die Anwendung der Methode erleichtern. Das Studium der Problemscharakteristiken hat sehr viel dazu beigetragen, dass ein einfacher und direkter Lösungsweg gefunden werden konnte.

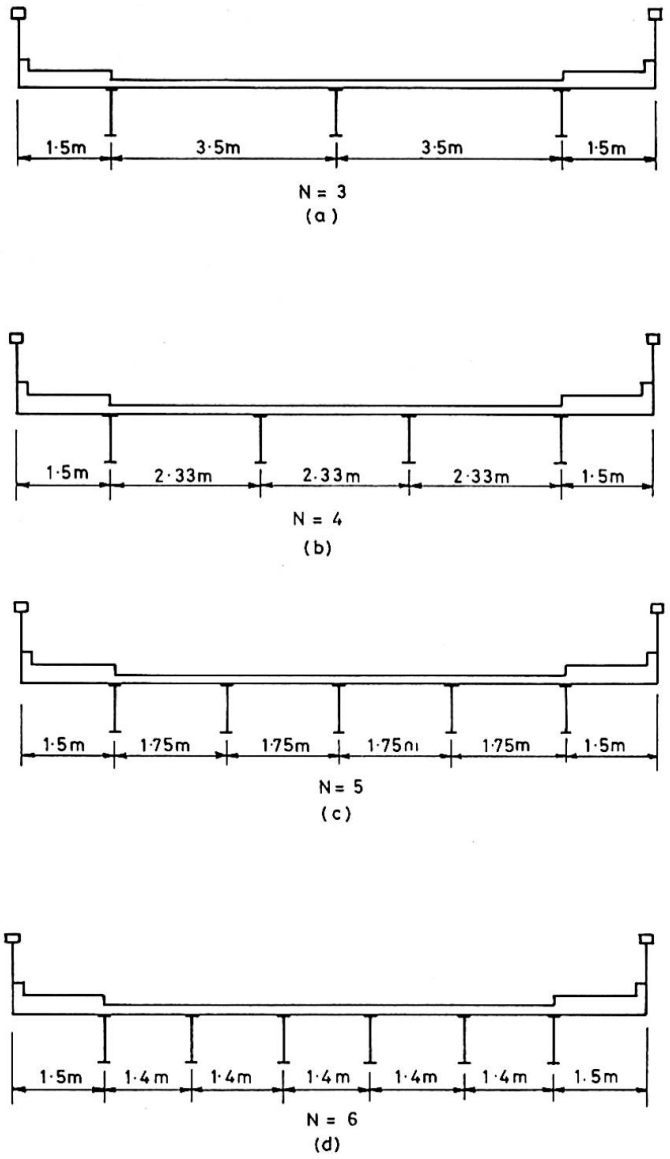

FIGURE 1

BRIDGE DECK CONFIGURATION

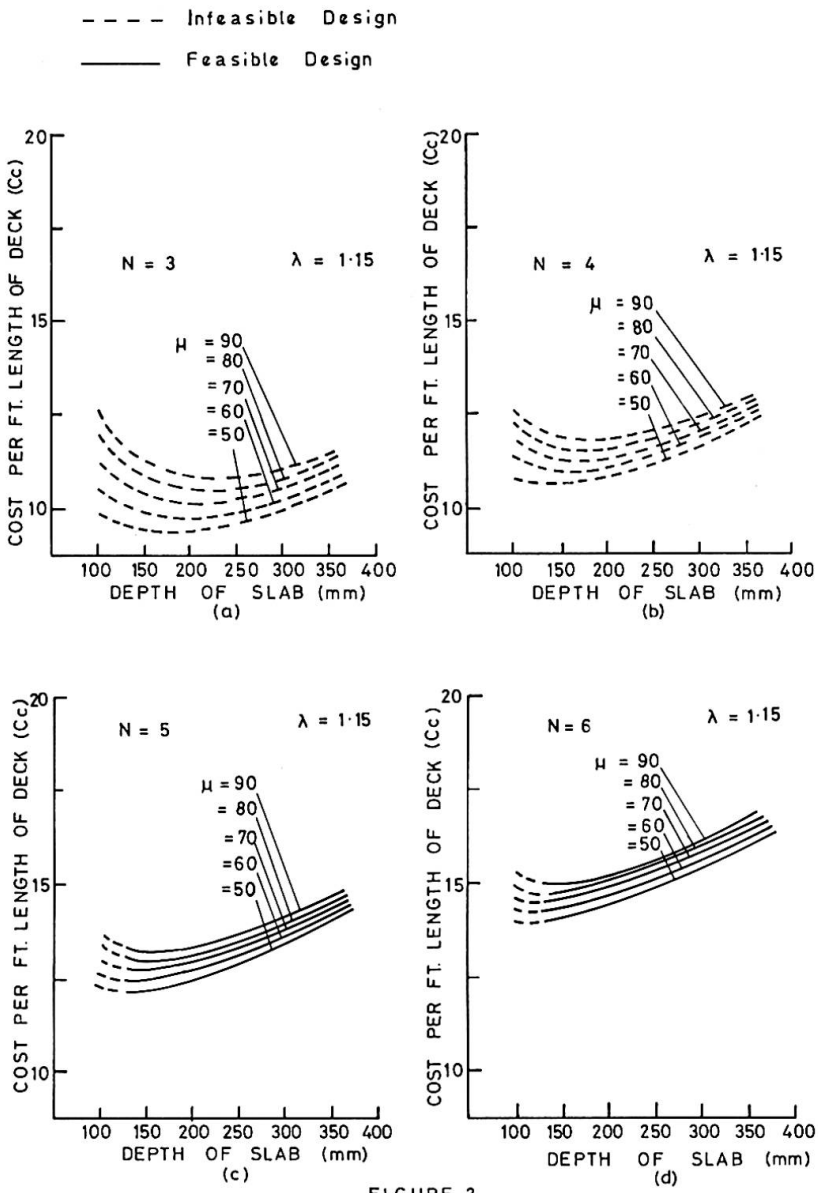

FIGURE 2

VARIATION OF DECK COST WITH DEPTH OF SLAB FOR DIFFERENT μ (762mm x 267mm x 194kg UB)

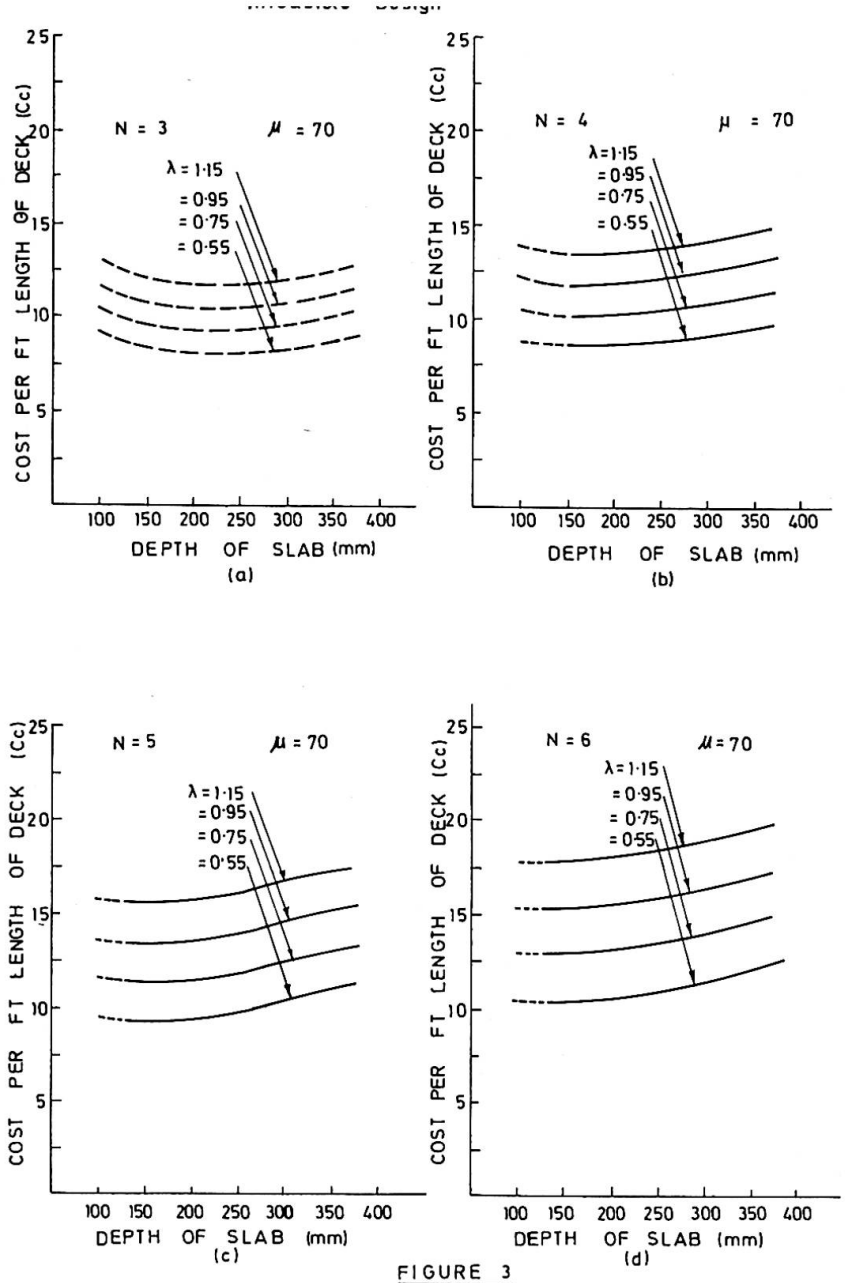

FIGURE 3

VARIATION OF DECK COST WITH DEPTH OF SLAB FOR DIFFERENT λ (914mm x 305mm x 224kg UB)

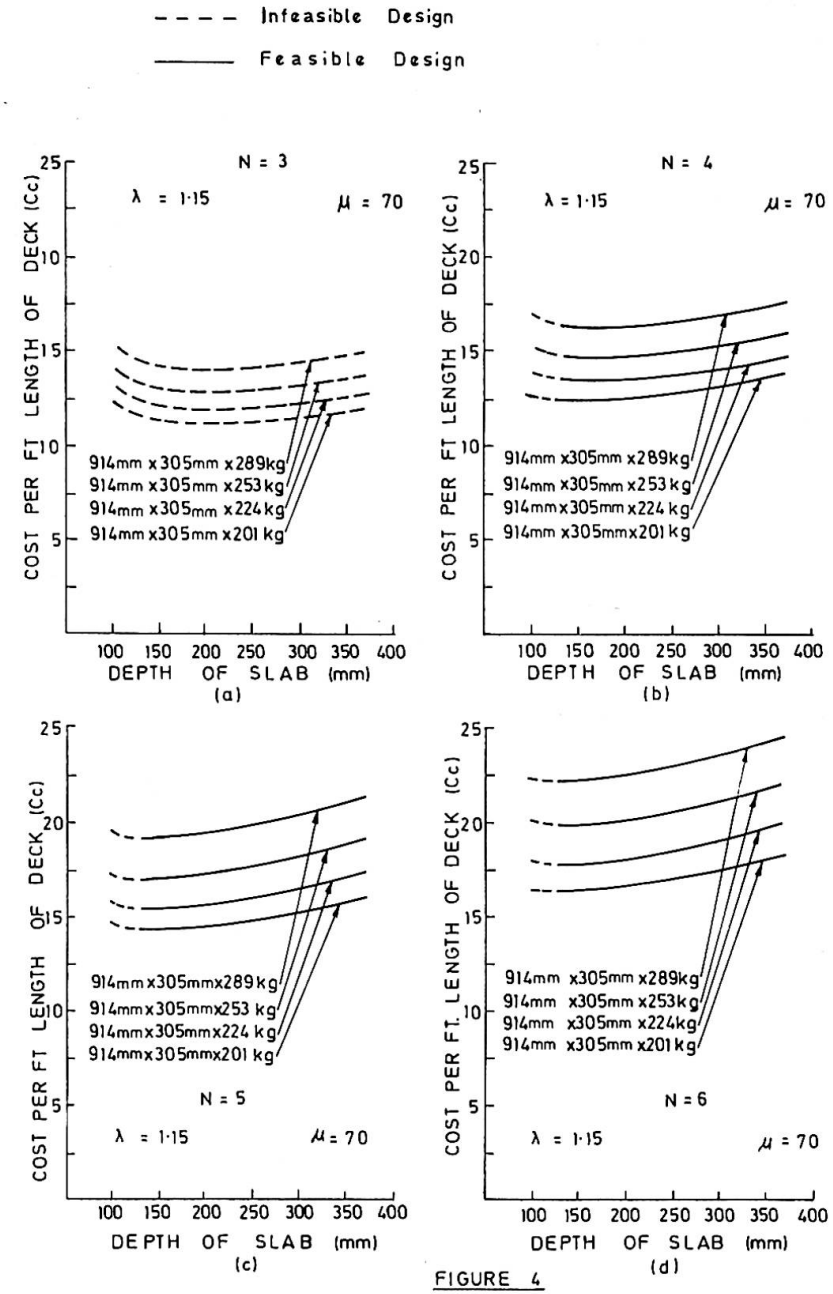

FIGURE 4

VARIATION OF DECK COST WITH DEPTH OF SLAB FOR GIRDERS OF SAME SERIAL SIZE

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Über die praktische Behandlung von Optimierungsaufgaben

On the Treatment of Optimization Problems for Better Use

Le traitement pratique des problèmes d'optimisation

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Mit Hilfe moderner Computer kann man heute ungleich komlexere Probleme angehen, als es zu den Zeiten der mechanischen Rechenautomaten möglich war. Diese Entwicklung ist natürlich zu begrüßen, leider hat sie auch eine Kehrseite. Neben den großen Vorteilen des Gebrauchs von EDV-Anlagen, wie

- hohe Kapazität
- schnelle Bearbeitungszeit
- Erledigung lästiger Routinearbeit

werden auch Nachteile sichtbar.

So verführt die Rechengeschwindigkeit der Automaten dazu, ein anstehendes Problem "simpel" zu lösen, z.B. unbekannte Funktionen durch einstellige Treppenfunktionen anstatt mehrstelliger Verfahren, welche bei finiten Elementen eine Menge von Unbekannten ausschalten, oder daß viele Optimierungsaufgaben durch ungezählte Iterationen gelöst werden, ohne sich ernsthaft mit der Möglichkeit einer direkten Lösung zu befassen.

Weiterhin wird durch die Maschinen die Suche nach einfacheren oder effizienteren Lösungswegen gebremst. Man hat ja ein Programm, wieso soll man nach einer neuen Methode zur Vereinfachung suchen?

So hängt dann leider nur zu oft die Arbeit des entwerfenden Ingenieurs von der Verfügbarkeit eines Programms, bzw. einer Rechenanlage ab. Nicht jeder ist aber in der glücklichen Lage, an ein Rechenzentrum angeschlossen zu sein. Es sollte daher, bei positivster Wertung der bisherigen Erfolge der EDV-Anlagen, nicht jener Grenzbereich vergessen werden, wo ein Problem nach entsprechender Behandlung auch mit einem Tischrechner, statt einer größeren Maschine gelöst werden könnte.

Bei solchen Forschungen kann der Ingenieur in den Entwurfsbüros am meisten profitieren. Er wäre dann imstande, noch mehr Probleme selbständig zu lösen, und nur noch bei wenigen Aufgaben auf große Rechenanlagen angewiesen.

An einem einfachen Beispiel soll demonstriert werden, wie sich aus einem anfänglichen Computerprogramm ein mit Tischrechnern lösbarer Algorithmus entwickelte.

Die Aufgabe ist es, einen Holz- Leimschichtträger als Einfeldträger so zu bemessen, daß er unter allen möglichen Rechteckquerschnitten die kleinste Fläche hat. Siehe Fig.1 u. Fig.2 .

Die Zielfunktion, sowie vier der Randbedingungen sind nichtlinear. In der ersten Bearbeitungsstufe wurde ein iteratives Programm mit Nullstellensuche nach Newton formuliert, welches den jeweils optimalen Punkt lieferte. Diesem Programm entspricht etwa die konventionelle zeichnerische Lösung. Siehe Fig.3 .

Zielfunktion:

$$Z: \quad F = b \cdot h = \alpha \cdot h^2 \longrightarrow \text{Minimum !}$$

Nebenbedingungen:

① $\quad \underline{\text{vorh } \tau - \text{zul } \tau \leqq 0} \longrightarrow \underline{\text{erf } h_1 = \sqrt[2]{qe \cdot l} \cdot P_1}$

② $\quad \underline{\text{vorh } \sigma - \text{zul } \sigma_B \leqq 0} \longrightarrow \underline{\text{erf } h_2 = \sqrt[3]{qe \cdot l^2} \cdot P_2}$

③ $\quad \underline{\text{vorh } f - \text{zul } f \leqq 0 \; (\text{zul } f = l/n_f)} \longrightarrow \underline{\text{erf } h_3 = \sqrt[4]{qe \cdot l^3 \cdot n_f} \cdot P_3}$

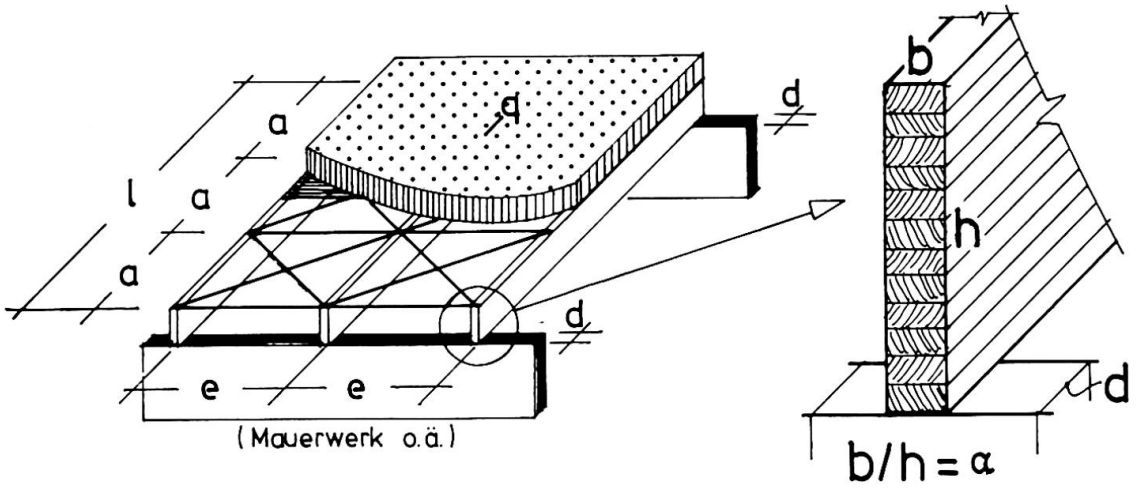
④ $\quad \underline{v_{kipp} \cdot \text{vorh } \sigma_B - \sigma_{kipp} \leqq 0 \; (\text{vorh } \alpha = l/n_a)} \longrightarrow \underline{\text{erf } h_4 = \sqrt[4]{qe \cdot l^3 \cdot \frac{1}{n_a}} \cdot P_4}$

⑤ $\quad \underline{\text{vorh}\sigma_{D\perp} - \text{zul } \sigma_{D\perp} \leqq 0} \longrightarrow \underline{\text{erf } h_5 = qe \cdot l \cdot \frac{1}{d} \cdot P_5}$

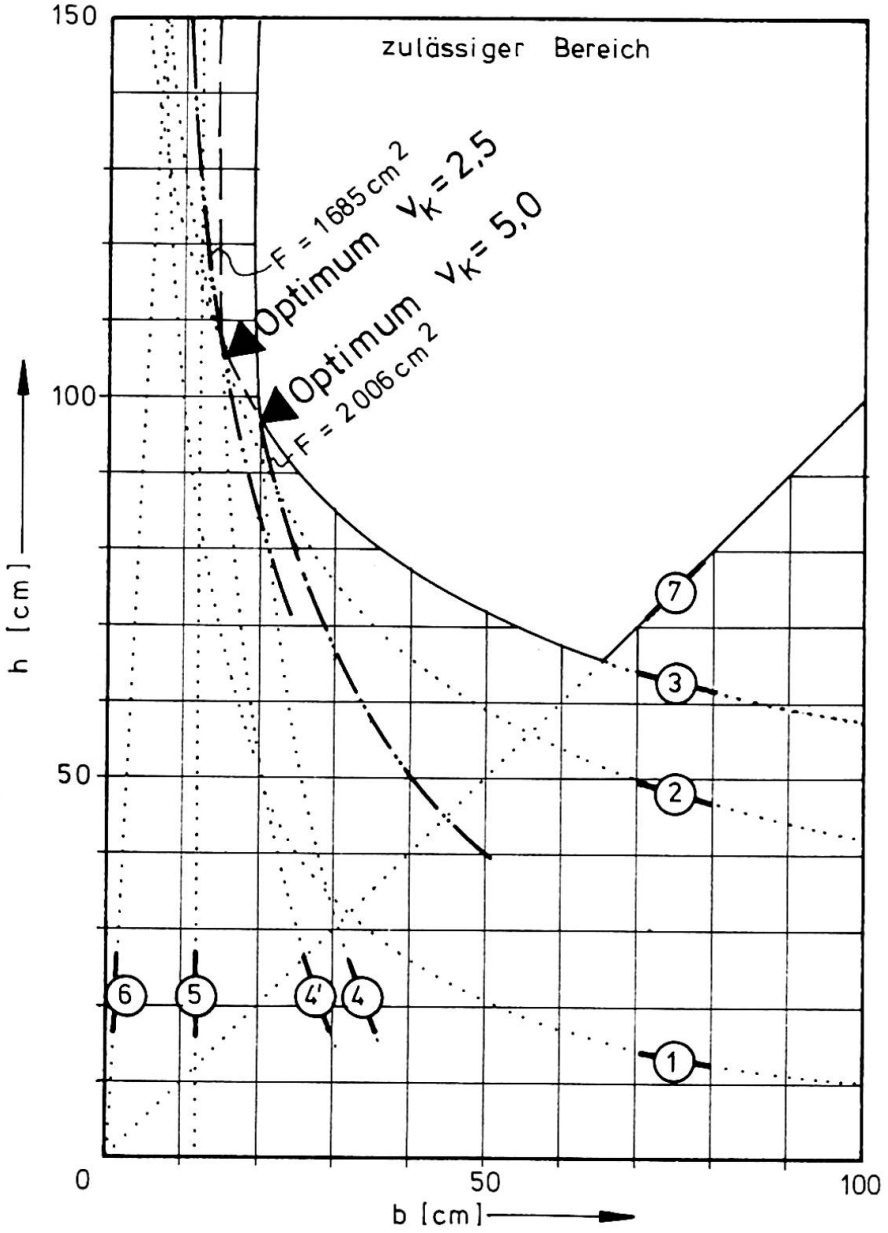
⑥ $\quad \underline{h/b - 25 \leqq 0} \longrightarrow \underline{\min \alpha = 0{,}04}$

⑦ $\quad \underline{1 - h/b \leqq 0} \longrightarrow \underline{\max \alpha = 1{,}0}$

Figur 1.: Zielfunktion und Nebenbedingungen



Figur 2.: Schema der Aufgabenstellung



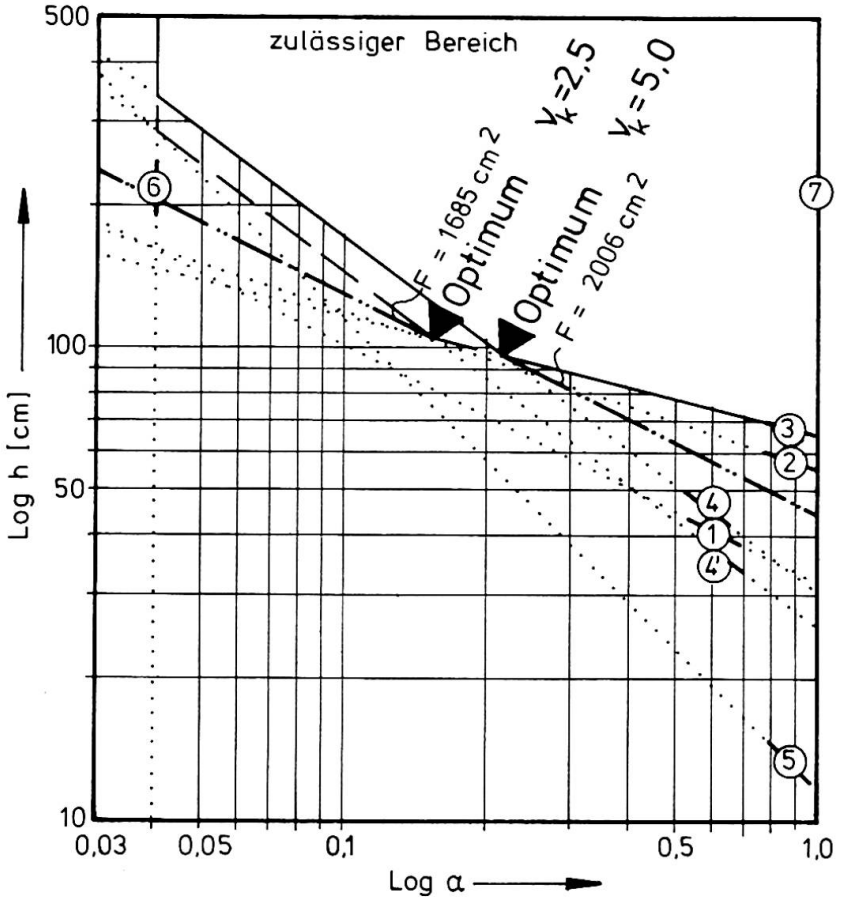
Figur 3.: Konventionelle zeichnerische Lösung

Das Beispiel ist für einen Brettschichtträger aus Nadelholz GKL I mit einer Spannweite von 20m, einer Last von 8,125kN/m, einer freien Kipplänge von 5m und einer zulässigen Durchbiegung von 1/2oo gerechnet.

Werden keine ungewollten Ausmitten in der Berechnung berücksichtigt, sollte man mit einer Kippsicherheit von 5,0 rechnen, anderenfalls mit der Kippsicherheit $\nu_{kipp} = 2,5$. Dies ist beim Gebrauch der Tabellen zu beachten.

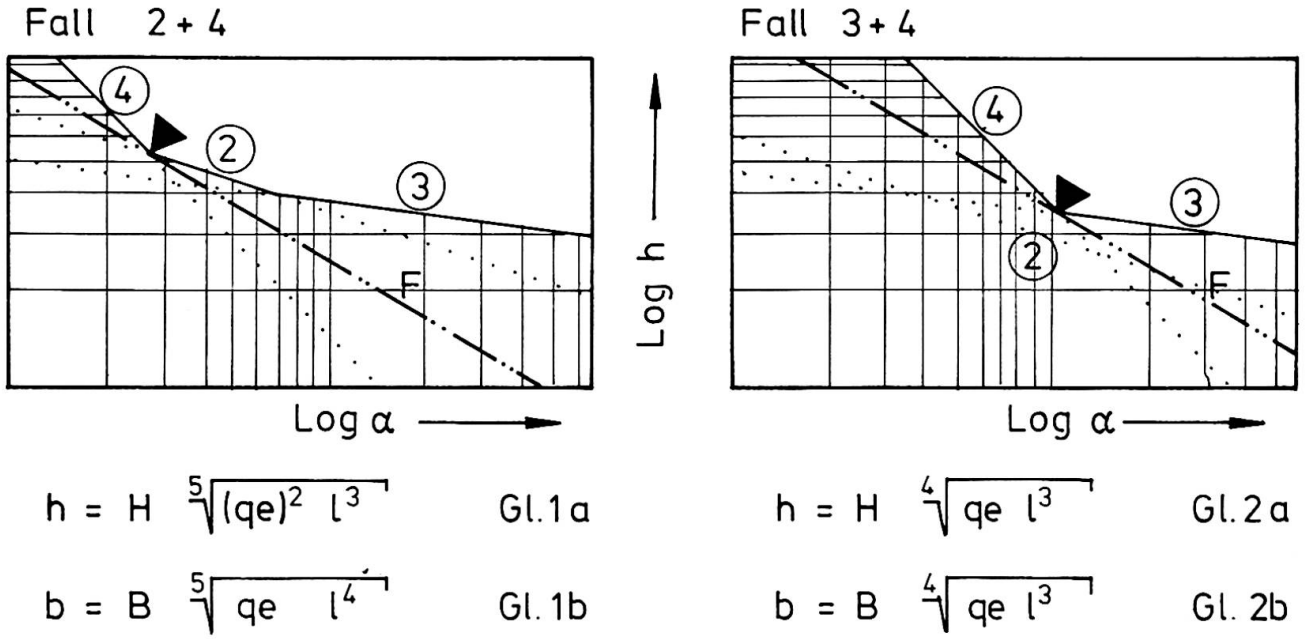
Die zeichnerische Darstellung ist umständlich und wegen der Nichtlinearitäten ungenau. Daher bestand die zweite Bearbeitungsstufe in der Einführung des logarithmischen Maßstabes, wodurch die Funktionen im praktisch vorkommenden Bereich linearisiert werden. So konnte ein Nomogramm entwickelt werden, welches die Suche nach dem optimalen Querschnitt innerhalb eines gegebenen Sortiments oder nach einer ganzzahligen Lösung sehr erleichtert.

Mit den Gleichungen aus Fig.2 und den Faktoren aus Tab.1 berechnet man für jede Randbedingung jeweils die erforderliche Querschnittshöhe für die Verhältnisse $b/h = \alpha = 0,03$ und $\alpha = 1,0$, trägt diese Werte auf Logarithmenpapier auf, verbindet zusammengehörige Punkte mit einem Lineal und hat so den Lösungsbereich. Die Zielfunktion F verläuft parallel zur Randbedingung (1) und wächst nach oben an. Siehe Fig.4. .



Figur 4.: Nomogramm

Für die Fälle, bei denen Ganzzahligkeit keine Bedingung ist und bei denen die Randbedingungen (1) und (5) vernachlässigbar sind, kann noch ein weiterer Bearbeitungsschritt gemacht werden. Durch Vereinfachung der Randbedingung (4) (Fehler auf der sicheren Seite und kleiner 5%) kann die kontinuierliche Lösung direkt angegeben werden, wenn festgestellt ist, welcher Fall maßgebend ist: 2+4 Biegespannung + Kippen, oder 3+4 Verformung + Kippen. Siehe Fig.5. .



$$h = H \sqrt[5]{(qe)^2\, l^3} \qquad \text{Gl.1a}$$

$$b = B \sqrt[5]{qe\, l^4} \qquad \text{Gl.1b}$$

$$h = H \sqrt[4]{qe\, l^3} \qquad \text{Gl.2a}$$

$$b = B \sqrt[4]{qe\, l^3} \qquad \text{Gl.2b}$$

Figur 5.: Unterscheidung praktisch vorkommender Fälle

Hierzu berechnet man nach Gl.1a und Gl.2a die für den jeweiligen Fall erforderliche Querschnittshöhe h. Der Fall mit dem größeren Wert für h ist maßgebend und optimal. Die zugehörige Querschnittsbreite errechnet sich dann nach Gl.1b bzw. Gl.2b .

Die Faktoren H und B sind in Tabelle 2 zusammengestellt. Setzt man die Spannweite in m und die Last in kN/m ein, ergeben sich die Querschnittsabmessungen in cm.

So ist aus einem Programm für einen Kleincomputer ein einfacher Algorithmus für die Praxis geworden.

Europ. Nadel-holz	GKL	P_1	P_2	P_3	P_4 v_{kipp}=2,5	P_4 v_{kipp}=5,0	P_5	α =
Kant-holz	I	2.887 16.667	3.864 12.436	1.118 2.686	2.304 31.956	2.739 38.002	0.025 0.833	1,0 0,03
	II		4.217 13.572					
	III		4.750 15.286					
Leim-quer-schnitt	I	2.500 14.434	3.770 12.132	1.092 2.623	2.276 31.577	2.707 37.552	0.025 0.833	1,0 0,03
	II		4.085 13.148					

Tabelle 1: Faktoren zum Nomogramm

n_a =		1	2	3	4	5	6	7	8	9	10	
2+4	I	4.913	5.644	6.121	6.483	6.779	7.031	7.251	7.447	7.625	7.787	H
		2.219	1.682	1.430	1.275	1.166	1.084	1.019	0.966	0.922	0.883	B
	II	5.678	6.523	7.074	7.493	7.835	8.126	8.380	8.607	8.812	8.999	H
		2.115	1.603	1.363	1.215	1.111	1.033	0.971	0.920	0.878	0.842	B
3+4	200	5.056	5.514	5.800	6.013	6.183	6.325	6.448	6.557	6.654	6.742	H
		2.198	1.695	1.456	1.307	1.202	1.123	1.060	1.008	0.964	0.927	B
	300	5.886	6.419	6.753	7.000	7.198	7.364	7.507	7.634	7.747	7.850	H
		2.089	1.611	1.384	1.242	1.143	1.067	1.007	0.958	0.917	0.881	B

Tabelle 2a: Faktoren H und B für eine Kippsicherheit von 5,0

n_a =		1	2	3	4	5	6	7	8	9	10	
2+4	I	5.644	6.483	7.031	7.447	7.787	8.076	8.329	8.555	8.759	8.945	H
		1.682	1.275	1.084	0.966	0.883	0.821	0.772	0.732	0.698	0.670	B
	II	6.523	7.493	8.126	8.607	9.000	9.334	9.626	9.887	10.122	10.338	H
		1.603	1.215	1.033	0.920	0.842	0.783	0.736	0.698	0.665	0.638	B
3+4	200	5.514	6.013	6.325	6.557	6.742	6.898	7.032	7.150	7.256	7.353	H
		1.695	1.307	1.122	1.008	0.927	0.866	0.817	0.777	0.744	0.715	B
	300	6.419	7.000	7.364	7.634	7.850	8.031	8.187	8.325	8.448	8.560	H
		1.611	1.242	1.067	0.958	0.881	0.823	0.777	0.739	0.707	0.679	B

Tabelle 2b: Faktoren H und B für eine Kippsicherheit von 2,5

ZUSAMMENFASSUNG

An einem einfachen Beispiel wird demonstriert, wie aus einem Computerprogramm ein mit Tischrechnern lösbarer Algorithmus entwickelt werken kann.

SUMMARY

It is shown, how from an initial computer program a simple algorithm for table-top calculators can be developed.

RESUME

On montre à l'aide d'un exemple simple, la façon dont un algorithme soluble par calculateur de poche a pu être développé à partir d'un programme d'ordinateur.

II

Synthesis and Conclusions

Synthèse et conclusions

Synthese und Schlussfolgerungen

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Several interesting points have emerged from the papers in the Preliminary Report and from the deliberations at this session. First of all it can be seen that work in Structural Optimization is being carried out at very many different technical levels. Some of the work which has been described can only be classified as purely theoretical research. Other papers have described attempts to apply some of the advanced methods of structural optimization to practical design problems. A few contributions have described purely practical approaches to structural optimum design which use none of the advanced methods now available.

This very wide range of papers demonstrates one common theme. As structural engineers we are all interested in designing better structures. The ultimate goal of any designer must surely be to produce the best possible design for a structure to fulfill prescribed tasks. We are all seeking the best solution and consequently our work itself is an optimum-seeking process.

There are probably many engineers today who believe that there is no necessity for designers to concern themselves with some of the complicated mathematical methods which have been developed for structural optimization. They will probably argue that a good designer with great experience will naturally produce good designs which are closely optimal, therefore there is no need to use complex numerical techniques. To these people I would like to say that fifteen years ago the same things were being said about the new computer-oriented analysis methods such as matrix methods and finite-element techniques. Now there are few projects which do not at some stage use computer-based analysis methods to verify the suitability of a design. The computer is now an established engineering tool and we should make maximum use of it to help in design. Some people seem to fear that the computer might take over entirely if it is allowed to enter into the design process thus reducing the designer to the status of a computer operator. This fear is totally irrational - the computer is merely a servant carrying out such tasks as are assigned to it. It has no intuition or innovative capacity and these are the essential attributes of a good engineer which cannot be reduced to mathematical form. In structural engineering the experience and skill of a designer will always be required. The increasing use of computers for optimum design can only add to the status of the designer by making his efforts consistently better.

It came as a pleasant surprise to me that IABSE had planned to run this structural optimization session at the Tokyo Congress. For the practicing engineer structural optimization is still largely a thing of the future. For twenty years structural optimization has been an area of intensive research and only recently, within the last five years or so, some

of this research has begun to percolate through to practical design engineers in a useable form. Some of this practical work we have seen during this session but the overwhelming tone of the discussions and papers has been that of research. The topic is not yet sufficiently developed for large scale practical use in everyday structural design. More research is necessary and more practical applications need to be studied in detail. Structural optimization still remains a thing for the future but the signs are that this future is not now very far away.

I cannot comment in detail on all this session's papers so I will restrict myself to comments upon a few. Firstly a very important point was made by Professor Massonnet on behalf of Dr. Maquoi. The use of the correct problem model is essential in optimization if realistic results are to be obtained quickly. It is seldom stressed that structural optimization is a two stage process. Firstly, a real-world problem must be modelled in mathematical form and secondly this mathematical problem must be solved. Often the first stage is the more difficult and, as Dr. Maquoi suggests, it is vitally important to get the right relationship between the real-world structure and its mathematical idealisation.

Dr. Hartmann's paper demonstrates this point very well. He shows a design for an encastré beam which, although optimal, is a totally impracticable design. Here the idealisation is incorrect and constraints to ensure a practical solution should have been included but were omitted. Unfortunately he ascribes the impracticable design to the use of mathematical optimization methods - the second stage of the process - and suggests that because mathematical methods produced this crazy design they should be discarded in favour of random search techniques. The logic here is surely wrong. It is not the mathematical techniques which are incorrect but the idealisation of the practical design problem

The papers by Di Carlo and Selleri are both interesting and worthy of further study. Mixed penalty functions as described by Di Carlo are not new, of course, but the specific application is interesting, demonstrating the existence of local optima. The problem of how to find the global optimum from several possible local optima is still an area for future research. Selleri's paper on cable structures and the papers by Daiguji and Feder on cable-stayed, prestressed structures reflect a growing interest in design problems associated with cables and similar work has been done on these topics in the USA and Great Britain.

Several of the papers presented for this Theme have used Linear Programming either straightforwardly or as a sequential iterative technique for design problems. To those people meeting structural optimization for the first time it might possibly appear that Linear Programming is a very powerful, widely applicable method for many structural design problems. I would like to set the record straight by stating that this is not so. Linear programming is a very simple method to use and consequently it is frequently employed to try to solve problems for which it is unsuitable. I will not repeat my warning as given in the Introductory Report, but will merely say that Linear Programming frequently causes more computational problems than it solves when applied to non-linear problems. Most structural optimization problems are non-linear.

I hope this session has given some idea of the great interest which now exists Worldwide in Structural Optimization. The Japanese National Group of IABSE quite rightly recognised the important place which structural optimization is likely to hold in structural design in the future and they showed remarkable foresight in suggesting it as a Theme for this Congress. I believe that at future Congresses much more will be heard of structural optimization and that in years to come it will become an established structural design tool. IABSE will then deserve much credit for stimulating discussions and interest in this topic at their 10th Congress.

III

Comportement des structures de bâtiments sous l'effet des incendies

Tragverhalten von Bauwerken unter dem Einfluss des Feuers

Behaviour of Building Structures under Fire Effects

III a

Effets thermiques des incendies dans les bâtiments

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Thermal Effects of Fire in Building

The Introductry Report of Subtheme III (a) "Thermal Effects of Fires in Buildings" written by us has been issued in 1975.

Four Preliminary Reports have been presented to the Subtheme III (a) and issued before the Congress.

In the first paper "Théorie des Equivalences" by Mr.E.Absi and M.Borensztein, CEBTP, France, the calculation method to predict the temperature-time field and the thermal stress in a concrete structural element are described and some calculation results are shown.

In the second paper "Détermination par la method des éléments fines des evolutions de temperature pour les structure soumises à l'incendie" by Mr.J.C.Dotreppe and M.Hogge, Universite de Liege, Belgique, the detail of a calculation method to predict the temperature-time field in a structural element is described and an experimental result is compared with the calculated one.

In the third paper "Application of a Limit State Concept to the Performance of a Structure under Fire Conditions" by Mr.H.L. Malhotra, Fire Research Station, U.K., a new concept considered some limit state to the performance of a structure, based on the semi-probablistic approach is described.

In the fourth paper "A Differentiated Approach to Structural Fire Engineering Design" by professor O. Pettersson, Lund University, Sweden, a pure engineering design method for fire resistance of building structure which has been legally available in Sweden is described.

These four papers are connected each other and I will explain as one story mixing with our introductry report.

As Mr.Malhotra describes in his paper, "the standard temperature-time curve" for the standard fire test of building structural elements was established, and the simple relationship between the fire load density and the necessary duration of fire

exposure along the standard temperature-time curve was derived by Dr.S.H.Ingberg about 60 years ago.

Then the concept of the fire resistance design for building structure was established.

That concept was to subdivide the building effectivelly into "fire resisting compartments," and the building regulations and codes designated the fire duration required, depending on the occupancies and the height or the size of building and also the fire resisting capacity of structural element was determined by the results of the full scale standard fire test.

In many countries except Sweden and France, this traditional design method has been still used legally and the International Standard Fire Test Method by ISO has recently been agreed.

Over the last fifteen years the fire research has been advanced remarkably, and the people intend to use the engineering method for fire resisting design apart from the traditional way.

The first country in which a new engineering method was permitted to use legally was Sweden. Professor Pettersson describes in his paper that it is necessary to define the following four items to establish an engineering design system.

a) the fire load characteristics,
b) the gas temperature-time curve of the fire compartment as a function of the fire load density, the ventilation characteristics of the fire compartment,
c) the temperature time field, and
d) the structural behaviour and minimum load bearing capacity of the fire exposed structure for a complete process of fire development.

Our introductry report has been described along these items. Now I explain some detail of each one.

a) The survey of the fire load of several occupancies has been done in several countries. But the fire load characteristics is slightly different for each country, because the structure and the use of building are different. Therefore it is necessary to determine the fire load density in each country, whatever the survey of fire load takes much labour and time.

The explanation of fire load density has many ways, depending on the purpose, as follows,

$$\frac{\text{equivalent weight of wood (Kg)}}{\text{unit floor area (m}^2\text{)}} \quad \text{(traditional)}$$

$$\frac{\text{potential heat content (Mj)}}{\text{unit floor area (m}^2\text{)}} \quad \text{(ISO)}$$

$$\frac{\text{effective heat content (Mj)}}{\text{unit interior surface area (m}^2\text{)}} \quad \text{(Sweden)}$$

$$\frac{\text{effective heat content (Mj)}}{\text{unit window area (m}^2\text{)}}$$

b) The gas temperature-time curve of a given compartment can be predicted roughly by the calculation of heat balance equation inside compartment, in which the heat release of fire load inside compartment.

In Sweden the calculated gas temperature time curve of a given compartment is taken as the heat load to the fire exposed structure, ignored "the standard temperature-time curve."

In Japan and other countries, the equivalent fire duration along the standard temperature-time curve, which temperature time area over the critical temperature of steel is same as the calculated one, is wanted to use as the heat load.

The result of the international corporative study of CIB W-14, in which eight laboratories were joined to test the model compartment fires, concluded that an experimental formula ($t_f = kL/\sqrt{A_W A_T}$) as shown in Malhotra's paper could be available to use for the estimation of fire duration alon the standard temperature time curve.

c) The temperature rise of steel member protected by the fire cover is not only depend on the thickness and the thermal properties of fire cover but also the heat capacity of steel member itself. It is not rational to determine only the thickness of covering material as usual designation in the codes and regulation.

In Sweden the thickness of fire cover is determined by the calculation of the heat conduction of each element exposed by each thermal load. Therefore a lot of tables and figures have been prepared, some of which are shown in Pettersson's paper.
In France the calculation method is applied legally in the concrete structural design since December 1975, which method is described in Adam's paper in subtheme III (c).

Because the theory of heat conduction has already been established, a lot of studies has been made for the prediction of temperature time field of structural elements exposed by fire.

In preliminary reports in this subtheme, Mr.Absi and Borensztein describe the calculation method used by the theory of equivalences and show some calculated results of concrete structural elements briefly. And Mr.Dotreppe and Hogge describe the detail of calculation method using the finite element one and compare an experimental result obtained from the heating of a block of concrete 18cm X 18cm with the calculated one.

In fact, in the calculation of heat balance equation in a compartment, the calculation of temperature time field of enclosed structual element is necessary. If a big computer could be used, the temperature time field could be calculated in the same time so that it is not necessary to divide b) and c).

Instead the theoretical calculation can be possible to any section of structual elements, the effect of spalling of concrete and of cracks of materials are rather difficult to insert the calculation and also the thermal properties of materials under the high temperature and the mechanism of moisture migration inside material under the high temperature are not well known, which are the problems to be solved.

d) The structual behaviour and the load bearing capacity of the fire exposed structure are the main themes of IABSE and especially in subtheme III (b) and III (c). In this subtheme III (a), Mr.Absi and Borensztein describe the calculation method of the thermal stress and the deformation used by the theory of equivalences, but the discussion of these problems is left to III (b) and (c).

For the systematize of the engineering design method based on from a) to b), many ways can be chosen. Now we look at the Swedish system as an example.

At the first, the rate of burning and the rate of heat release in a given compartment are calculated from the fire load density, the size and the geometry of compartment and the ventilation characteristics. Then the gas temperature time curve is calculated which is assumed as the heat load.

Input this heat load to the structural element, the temperature-time field inside it is calculated.

From this temperature-time field and the restraint forces, moments, thermal stress and the reduction of mechanical properties in the structual element, the load carring capacity Rd is calculated.

Leaving these calculated results, the load effect at fire condition Sd is calculated separately.

If Sd is greater than Rd, the proposed structure is modified and recalculation is made until to obtain the satisfaction of $Sd<Rd$.

In Japan, tall apartment buildings of a big project was designed their fire resistance used by a similar engineering method, but it was specially permitted, not available to use any building.

Finally, the new concept of the limit state approach in fire by Mr.Malhotra is introduced. He divides the limit state into two, one is the limit state of stability which means the structural load carring capacity under fire based on the probablistic approach and another is the limit state of integrity which means the capacities of barrier of the compartmentation.

A similar probablistic analysis for steel structure is presented by Dr.S.E.Magnusson in subtheme III (b).

These studies based on the probablistic approach are only started, but the re-consideration of fire safety will become an important research subject in the fire engineering field because it would be the important basic concept of engineering design.

Numerical Calculation of the Temperature Distribution in Hot Gas Plume from a Window

Calcul numérique de la distribution des températures dans une colonne de gaz chaud s'échappant d'une fenêtre

Numerische Berechnung der Temperaturverteilung in einer aus einem Fenster ausströmenden Heissluftsäule

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1. INTRODUCTION

In order to prevent the fire spread to upstairs through broaken windows in a building,it is necessary to provide a fireproof spandrel between the windows. The estimation of its necessary height was once studied by YOKOI*[1],who conducted both dimensional analysis and small scale experiments on the behavior of the hot gas plume ejected from a window to find that the temperature distribution along its trajectory would be estimated by the normalized temperature Θ which is defined as

$$\Theta \equiv \Delta\theta r_0^{5/3} / \sqrt[3]{Q^2\theta_0/C_p^2\rho^2 g} \quad (1)$$

where $\Delta\theta$:excess temperature of the trajectory(°C), Q: released energy rate from window(kcal/sec), θ_0:temperature of ambient air(°K), C :specific heat of air at constant pressure(kcal/kg°K) r_0:equivalent radius of window(m) and ρ:density of hot gas(kg/m^3). Fig.1 summarizes the results of the small scale experiments for various geometries of window. This simple method gives good results for many cases, but when the refractoriness of a spandrel or the interaction between the spandrel and the plume is discussed, it will be also necessary to predict the temperature dis-

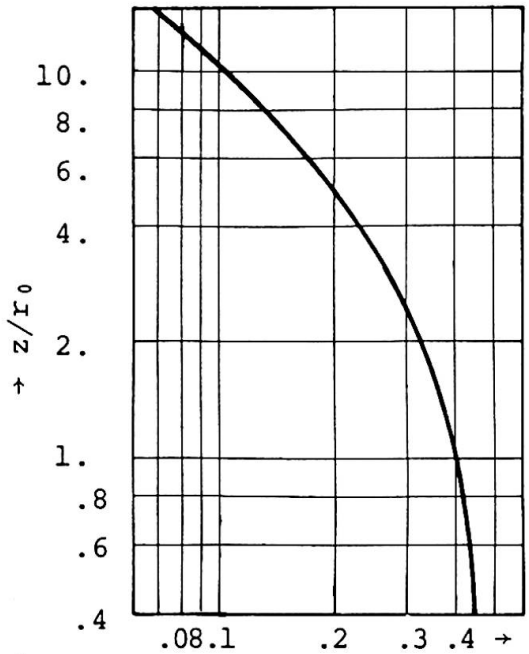

Fig.1 TEMPERATURE DISTRIBUTION ALONG THE TRAJECTORY OF HOT GAS PLUME*[1]

tribution in hot gas plume,especially near the spandrel above the window.

In this paper,we introduce the calculation method by the finite difference approximation of the governing differential equations on the behavior of hot gas plume and then show a calculation result with an experimental one carried out under the almost similar condition to the one for the numerical calculation.

2. GOVERNING EQUATIONS

The governing equations of a thermally expanding or contracting turbulent motion of gas are the following set of 6*[2].

EQUATION OF MOMENTUM:

$$\frac{\partial \overline{\rho u}_i}{\partial t} + \frac{\partial \overline{\rho} \overline{u}_i \overline{u}_j}{\partial x_j} \simeq -\frac{\partial \overline{P}}{\partial x_i} + \frac{\partial}{\partial x_j}\{\overline{\sigma}_{ij} + K(\frac{\partial \overline{\rho u}_i}{\partial x_j} + \frac{\partial \overline{\rho u}_j}{\partial x_i})\} - \delta_{i3}\overline{\rho} g \qquad (2)$$

where u:velocity(m/sec), P:pressure(kgm/sec^{2}m^2), σ:viscosity stress(kgm/sec^{2}m^2), K:eddy coefficient(m^2/sec), i or j:tensor mark and δ:KRONECKER delta. Overbarred quantities denote the time-smoothed variables. The time-smoothed velocity is given by

$$\overline{u}_i \simeq \frac{\overline{\rho u}_i + K(\frac{\partial \overline{\rho}}{\partial x})_i}{\overline{\rho}} \qquad (3)$$

EQUATION OF CONTINUITY:

As the time-smoothed density is obtained from the equation of state in our calculation scheme,the equation of continuity should be transformed to the POISSON type equation for $\overline{P}$. This equation is obtained by substituting the equation of momentum into the natural form of continuity equation $\partial\overline{\rho}/\partial t + (\partial\overline{\rho u}/\partial x)i = 0$.

$$\frac{\partial^2 \overline{P}}{\partial x_i^2} \simeq \frac{\partial^2 \overline{\rho}}{\partial t^2} + \frac{\partial^2}{\partial x_i \partial x_j}\{\overline{\sigma}_{ij} + K(\frac{\partial \overline{\rho u}_i}{\partial x_j} + \frac{\partial \overline{\rho u}_j}{\partial x_i}) - \overline{\rho} \overline{u}_i \overline{u}_j\} - g\frac{\partial \overline{\rho}}{\partial x_3} \qquad (4)$$

CONSERVATION EQUATION OF ENERGY:

$$\overline{\rho}\frac{\partial \overline{h}}{\partial t} + \frac{\partial \overline{\rho u}_i \overline{h}}{\partial x_i} \simeq \frac{\partial}{\partial x_i}(\kappa\frac{\partial \overline{T}}{\partial x_i} + K\frac{\partial \overline{h}}{\partial x_i}) + \overline{Q} + h\frac{\partial \overline{\rho u}_i}{\partial x_i} \qquad (5)$$

where h:enthalpy of air(kcal/kg), κ:thermal conductivity(kcal/sec m) and Q:generation rate of heat(kcal/secm^3). The dissipation of kinetic energy is ignored for its minor role in the conservation of energy.

DEFINITION OF EDDY COEFFICIENT:

PRANDTL's dimensional relationship is applied to the modeling of eddy coefficient. In this model,eddy coefficient is given by

$$K \simeq \sqrt{\overline{q}}\,\ell \qquad (6)$$

where the mixing length 'ℓ' may be determined geometrically and the turbulent energy is given from the conservation equation for it.

$$\frac{\partial \overline{\rho} \overline{q}}{\partial t} + \frac{\partial \overline{\rho u}_i \overline{q}}{\partial x_i} \simeq \frac{\partial}{\partial x_i}(\overline{\rho} K\frac{\partial \overline{q}}{\partial x_i} + \mu\frac{\partial \overline{q}}{\partial x_i}) + K\frac{\partial \overline{\rho}}{\partial x_3} g - \overline{\rho\varepsilon} + K(\frac{\partial \overline{\rho u}_i}{\partial x_j} + \frac{\partial \overline{\rho u}_j}{\partial x_i})\frac{\partial \overline{u}_j}{\partial x_i} \qquad (7)$$

where μ:dynamic viscosity(kg/secm) and ε:turbulent energy decay rate(m^2/sec^3).

EQUATION OF STATE:

The change or variation of the pressure in fire is usually so small that the equation of state may be approximated correctly by

$$\bar{\rho} \simeq \frac{P_0}{R\bar{T}} \qquad (8)$$

where P_0:referential pressure(kgm/sec^{2}m^2), R:gas constant and T: absolute temperature of air(°K).

3. OUTLINE OF THE CALCULATION METHOD

The governing equations above introduced are of course so complicated for an analytical solution that we solve them simultaneously in a numerical manner. However,because we do not have enough space to describe everything about the calculation method, we present the outline of the finite difference calculation method which is detailed in REF.2.

In our numerical code,the unsteady equations (2),(3) and (7) are transformed into explicit finite difference equations, whilst the POISSON type equation for the pressure (4) is solved by an iterative method such as Over-Relaxation method. For the finite difference calculation of the spacially differentiated terms,the Cell Model in which the physical variables are arranged by the way as shown in Fig.2 is used. A stable marching of the unsteady calculation is achieved only when the finite increment of time satisfies the following criterion which is known as COURANT-FRIEDRICHS-LEWY condition.

$$\Delta t < \frac{1}{\left(\sum_i^3 |\bar{u}_i/\Delta x_i|\right)_{max}} \qquad (9)$$

Then,the finite difference equations for (2)∿(8) are solved unsteadily by the procedure as shown in Fig.3.

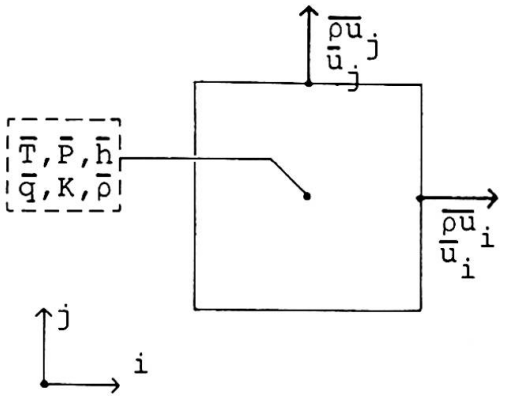

Fig.2 GRID ARRANGEMENT IN CELL MODEL

4. CALCULATION OF THE TEMPERATURE DISTRIBUTION IN HOT GAS PLUME

Now,we apply the calculation method above introduced to the estimation of the temperature distribution of a hot gas plume ejected from a broaken window.

It was suggested by YOKOI*[1] on the basis of the results of his small scale experiments that the hot gas plume from a wide opening would close to the wall or spandrel above it to be a possible cause of the fire spread to upstairs. This phenomenon which may be interpreted by the COANDA effect was also observed in the full scale experiment conducted recently by KAWAGOE et al*[3]. Such a phenomenon seems to be represented pertinently in a two-dimensional calculation,because we may conceptually replace a wide opening by a two-dimensional one without bringing any serious problem.

In this paper,we make a numerical calculation of the hot gas plume under the conditions which are determined according to KAWA-

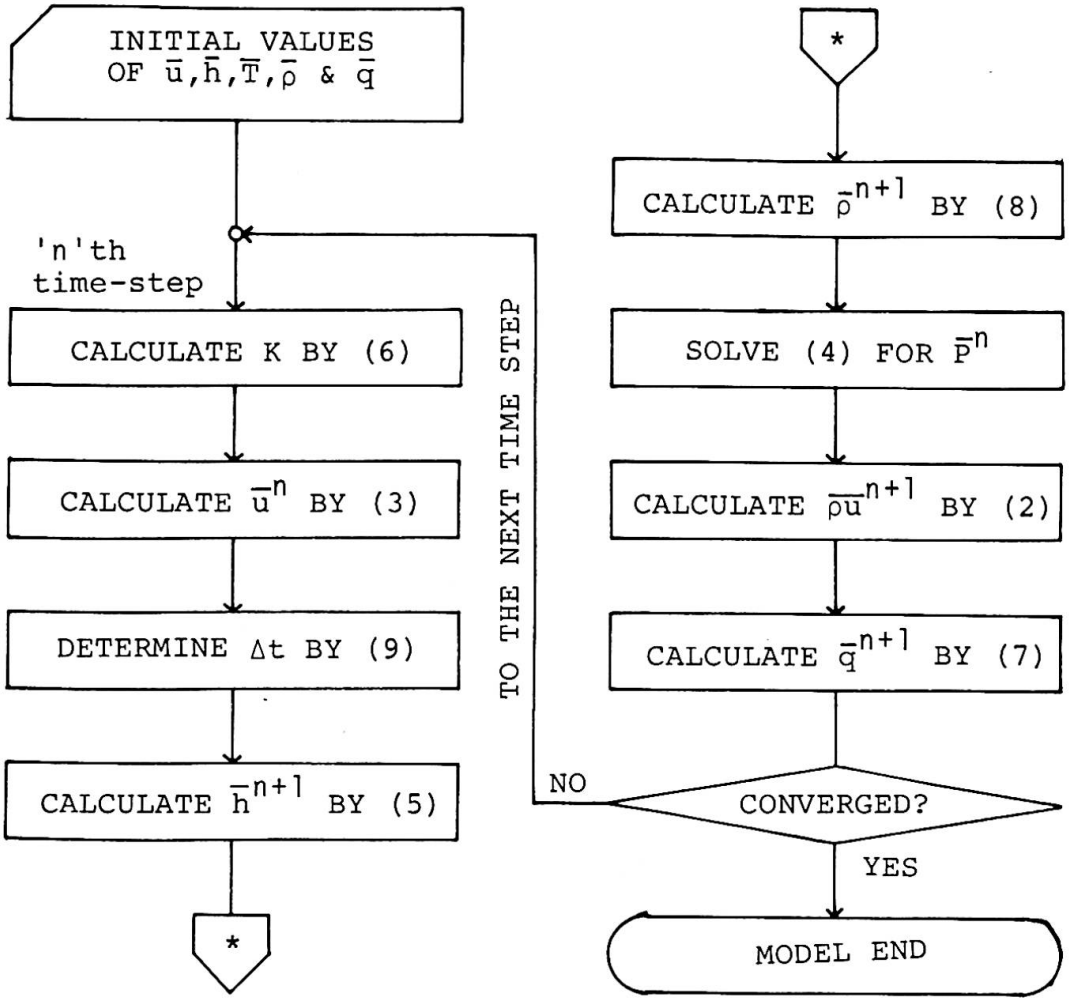

Fig.3 OUTLINE OF THE UNSTEADY CALCULATION PROCEDURE

GOE's experiment as shown in Tab.1 and then compare the calculation result with the experimental one. Among these conditions for the calculation,the height of neutral plane and the distribution of the mixing length are determined empirically and the spouting velocity is estimated by YOKOI's equation.

Tab.1 CONDITIONS FOR THE NUMERICAL CALCULATION

TEMPERATURE OF HOT GAS PLUME AT OPENING	900°C
TEMPERATURE IN UPSTAIRS ROOM	100°C
TEMPERATURE OF AMBIENT AIR	30°C
MASS FLUX RATE OF PLUME AT OPENING	$\rho U = \sqrt{2gh''(\rho_0 - \rho)}$ ρ_0:density of ambient air h":height from neutral plane
MIXING LENGTH	$\ell = 0.4y$, $\ell_{max} = 0.2H$ y:distance from boundary H:height of building

In Fig.4 are superimposed the calculated isotherms on the temperature field generated from the experiment. Although the discrepancy between the calculated and experimental distributions near the opening of the fire compartment is considerably great,the phenomenon above mentioned which may be a cause of the fire spread to upstairs is represented pertinently in the calculation result.

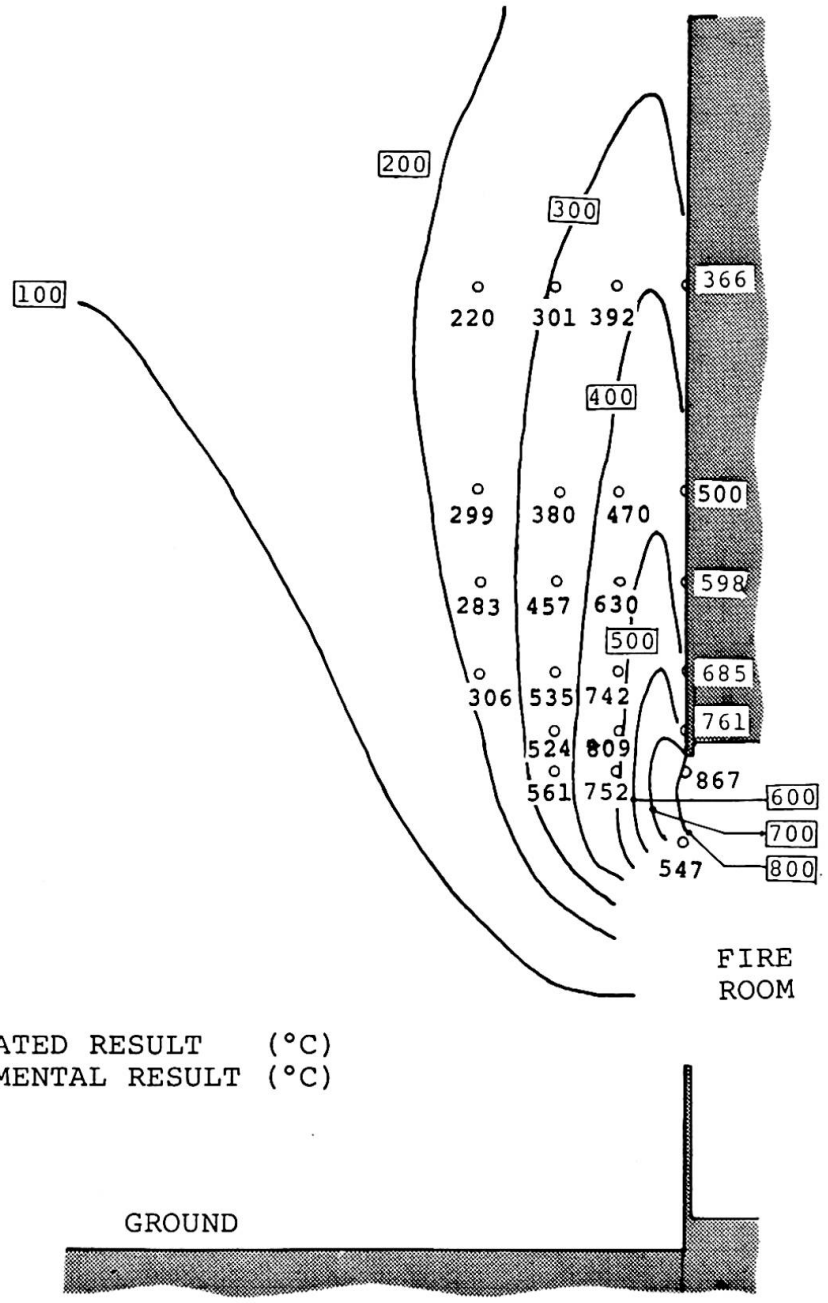

Fig.4 TEMPERATURE DISTRIBUTION IN HOT GAS PLUME FROM A WINDOW (COMPARISON OF CALCULATED AND EXPERIMENTAL RESULTS)

5. CONCLUSIONS

The temperature distribution in two-dimensional hot gas plume was calculated as an application of the prediction method. The use of a large EDPS will permit also a three-dimensional calculation, because the calculation method for the three-dimensional case does

not differ basically from the one for the two-dimensional case which is introduced in this paper. Though some problems such as the calculation of the flame radiation are left as future subjects, we can utilize such a calculated temperature distribution for designing the height or refractoriness of a spandrel from the view point of fire safety.

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SUMMARY

This paper presents a method for the numerical calculation of the temperature distribution in a hot gas plume from a window. The calculation is given for a two-dimensional window and also compared with experimental results. Calculation and experiments show a good agreement.

RESUME

Cet article présente une méthode numérique pour calculer la distribution des températures dans une colonne de gaz chaud s'échappant d'une fenêtre. Le calcul est présenté pour une fenêtre à deux dimensions et est aussi comparé avec des résultats expérimentaux. La concordance entre le calcul et l'expérience est bonne.

ZUSAMMENFASSUNG

Der Autor beschreibt ein numerisches Verfahren zur Berechnung der Temperaturverteilung in einer aus einem Fenster ausströmenden Heissluftsäule. Die Berechnung wird für ein zweidimensionales Fenster vorgelegt und mit Versuchsergebnissen verglichen. Die Uebereinstimmung zwischen Berechnung und Versuch ist gut.

Design of Tall Apartment Buildings for Fire Resistance

Conception de bâtiments d'habitation hauts en vue de leur résistance à l'incendie

Entwurf von hohen Wohnhäusern in bezug auf Brandeinwirkungen

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The existing Japanese building codes for fireproof constructions prescribe the fire endurance of the individual structural elements, such as beams, columns, floors and so on, without concerning the fire properties and structural features of buildings. Buildings have been designed against fire according to these codes. However, researches on the fire properties in compartments and the behaviour of steel structures under fire have recently been carried out and made the design of fireproof constructions possible by engineering methods.

This paper discusses the method and the results of the fireproofing design of the Ashiyahama Housing Project currently under construction. The Ashiyahama Housing project consists of many multistoried buildings as shown in Photo 1 and the dwellings totaled 3384.

The fireproofing design system used on this project is illustrated in Fig.1, and the following procedure was applied:

1) Fire compartments were allocated as shown in Fig. 2.
2) Fire loads were estimated from the quantity and the quality of the combustible materials for individual fire compartments.
3) Using a monograph, the equivalent fire duration time of dwelling unit was obtained from the fire load, floor factor, temperature factor and others. The equivalent fire duration time of public floor was determined by taking into account the fire load and burning phenomenon in the open space.
4) If the fire was not to spread to the adjoining fire compartments, then advance to the next step.
5) Using a computer program, allowable temperature of the steel frame was calculated from dead load, live load, thermal expansion and deterioration of steel members at elevated temperature. Shown as example, Fig.3 represents the joint displacement according to only thermal expansion and Fig.4 shows the distribution of bending moment that is due to the ordinary design load and the thermal expansion.

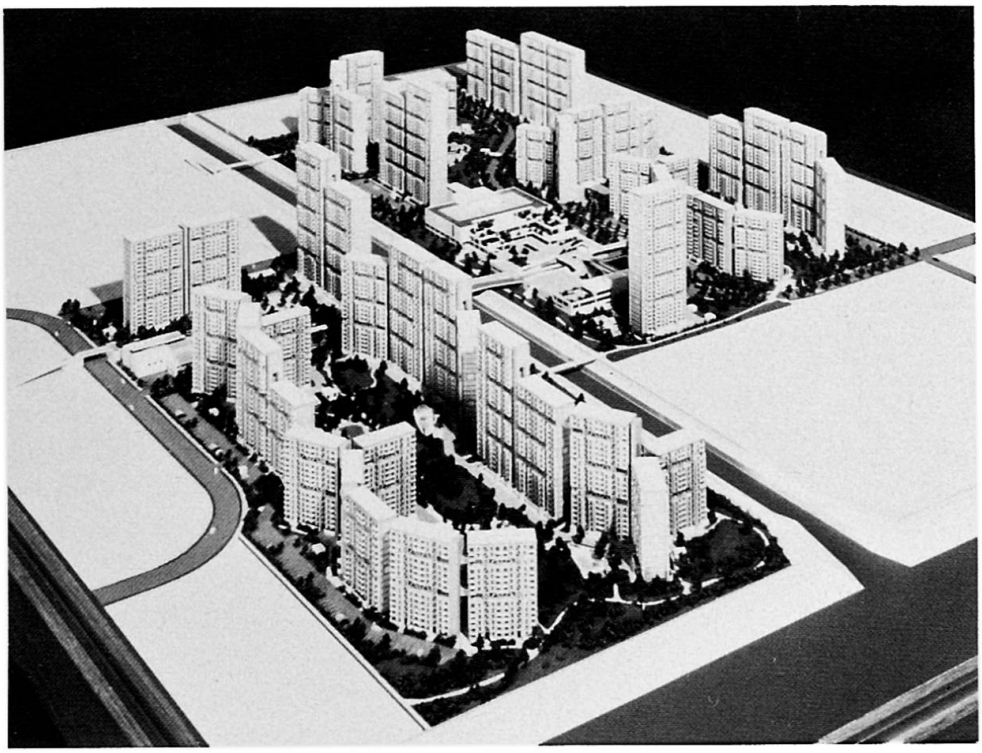

Photo 1: The bird's-eye view of the Ashiyahama Housing Project.(model)

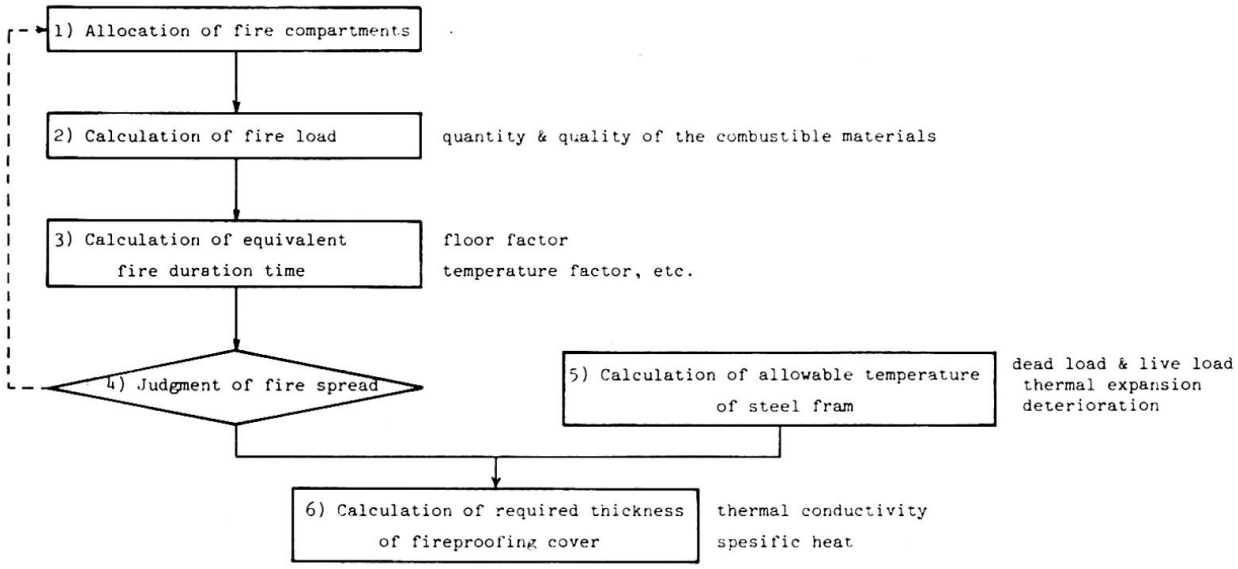

Fig. 1: The fire proofing design system

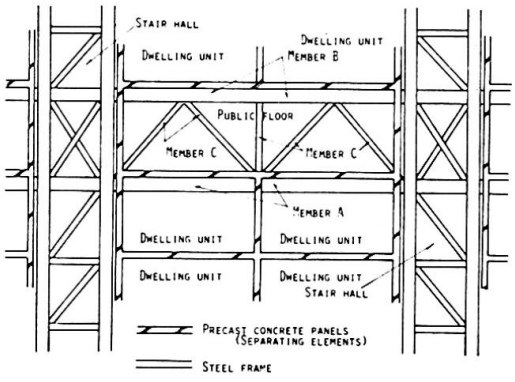

Fig. 2: The fire compartment and steel frame

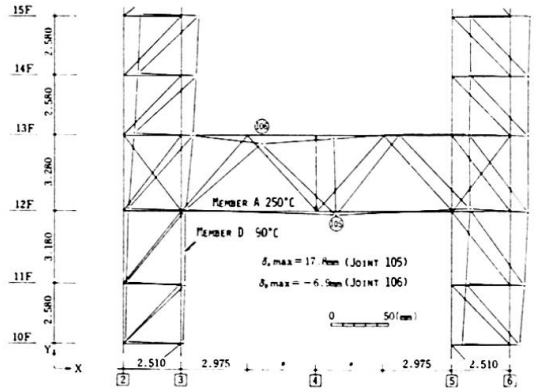

Fig. 3: Joint displacement according to only thermal expansion

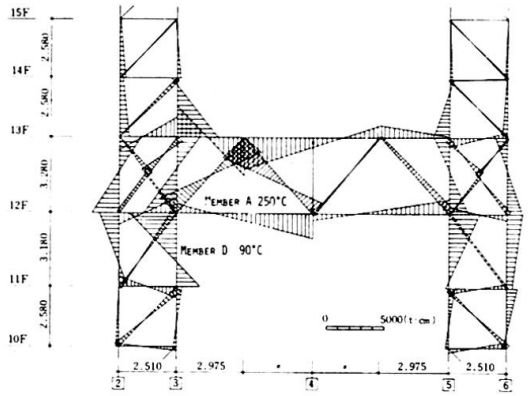

Fig. 4: Distribution of bending moment according to the ordinary design load and the thermal expansion

Fire compartments	Fire load (Kg/m²)	Equivalent fire duration time (min.)	steel members in compartment	allowable temperature of steel members (°C)	Required thickness of fireproofing cover* (mm)
Dwelling unit	60	90	A	250	45
Public floor	15	30	B	200	15
			C	100	15**
Stair hall	0		others		0

* Sprayed asbeston.

** This value was determined by taking into account the heating condition and burning phenomenon in the open space.

Table 1: Results of the fireproofing design

6) The need for some fireproofing cover was realized by which the temperature rise of steel member could be kept below the allowable temperature after heating for equivalent fire duration time along the standard time-temperature curve. The required thicknesses of the fireproofing cover for members named A, B and C were calculated. It was found that other members did not require fireproofing cover.

These results are tabulated in Table 1.

Adopting the fireproofing design in this project despite the current building codes, structural safety against fire was secured and economical design was established.

The fireproofing design system adopted in this project can be applied, without modification, to such design of other buildings.

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SUMMARY

This paper discusses the method and results of the fireproofing design for the Ashiyahama Housing Project currently under construction. By adopting the fireproofing design in this project, without restriction of the existing Japanese building codes for fireproofing construction, the structural safety against fire was secured and economical design was satisfied.

RESUME

Cette étude discute la méthode et les résultats du calcul et de la conception du quartier d'habitation d'Ashiyahama en vue de la résistance à l'incendie. L'adoption de cette méthode qui n'est pas en contradiction avec la réglementation actuelle japonaise en la matière a permis d'assurer la sécurité structurale contre l'incendie et de réaliser un projet économique.

ZUSAMMENFASSUNG

Dieser Artikel erörtert die Methoden und Ergebnisse des Entwurfs in bezug auf Brandeinwirkungen für das derzeitig im Bau stehende Ashiyahama Wohnbauprojekt. Die für dieses Projekt herangezogene Methode gewährleistet einen ausreichenden Brandschutz, ist zudem wirtschaftlich und entspricht in allen Punkten den bestehenden japanischen Vorschriften für brandsichere Bauten.

Calcul automatique de la résistance au feu des ossatures métalliques

Automatische Berechnung des Feuerwiderstandes von Stahlbauten

Automatic Computation of Fire Resistance of Steel Structures

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1. INTRODUCTION.

Dans cet article, on présente un code de calcul permettant d'évaluer, de manière automatique, la résistance au feu des ossatures métalliques protégées et non protégées.

Le comportement au feu des structures métalliques présente un certain nombre de caractéristiques qui permettent l'utilisation de techniques de calcul particulières, analogues à celles utilisées pour le calcul à la ruine des structures métalliques ordinaires. Le procédé s'apparente aux méthodes de l'analyse limite des ossatures en acier doux, basées sur le concept de rotule plastique.

Le calcul de l'évolution de la température dans les éléments se fait de manière simple en admettant que la température est uniforme sur la section droite.

2. CALCUL DE L'ELEVATION DE LA TEMPERATURE DANS LES PROFILS.

Dans le Rapport Préliminaire [5], on a indiqué la technique utilisée pour le calcul de l'évolution de la température dans les éléments soumis à l'incendie. En règle générale, il s'agit de résoudre l'équation de Fourier, assortie de conditions aux limites de différents types.

Dans le cas d'un profilé en acier, le calcul des températures peut être fortement simplifié, par suite des valeurs élevées de la conductivité thermique λ . L'hypothèse simplificatrice consiste à admettre que la température du profilé est uniforme. Cette hypothèse est plausible dans le cas des profilés ordinaires en double tê, par suite de la minceur de l'âme et des semelles et de la grande surface exposée au feu.

2.1. Cas d'un profilé non protégé.

Soit un élément de volume dV soumis à un flux de chaleur Q. Le bilan calorifique de cet élément s'écrit (cf. figure 1) :

$$Q\, dA = c\rho \frac{\partial \theta}{\partial t} dV \qquad (1)$$

où dA est la surface extérieure de l'élément.

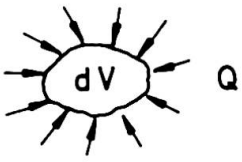

Figure 1

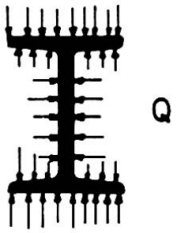

Figure 2

Le même bilan peut s'écrire en ce qui concerne un profilé métallique non protégé, puisqu'on suppose que les calories absorbées se répartissent instantanément de manière uniforme dans toute la masse de l'acier (figure 2). L'équation (1) s'écrit alors :

$$Q \frac{U}{F} = c\rho \frac{\partial \Theta}{\partial t} \qquad (2)$$

dans laquelle apparaît le facteur de massiveté $\frac{U}{F}$.

En passant aux différences finies et en explicitant la densité de flux de chaleur Q, on obtient une équation de résolution permettant de calculer pas à pas la courbe de montée en température du profil [3] [4].

2.2. Cas d'un profilé protégé.

Le calcul de la réponse thermique des structures protégées peut être effectué à différents degrés de raffinement.

La méthode la plus simple envisage un bilan thermique restreint ne concernant que l'acier, dans lequel U est la surface intérieure de l'isolant qui constitue, pour le profilé, la surface émettrice. La massiveté $\frac{U}{F}$ devient donc dépendante du type de protection.

Dans ce cas, il est nécessaire de faire un certain nombre d'hypothèses concernant la répartition de la température dans l'isolant pour calculer les échanges et les propriétés thermiques moyennes du matériau de protection. On peut alors résoudre l'équation obtenue par itérations comme précédemment.

De telles méthodes ne fournissent des résultats satisfaisants que dans le cas de produits secs et pour des épaisseurs de protection relativement faibles. Pour des problèmes où l'inertie thermique de l'isolant n'est pas négligeable, il faut s'orienter vers des méthodes plus raffinées.

Il existe certaines méthodes intermédiaires, comme celle qui consiste à écrire le bilan thermique de la manière suivante :

$$Q\,U = c_a \rho_a \frac{\partial \Theta_a}{\partial t} F + c_i \rho_i \frac{\partial \Theta_i}{\partial t} L_i e_i \qquad (3)$$

où a se rapporte à l'acier et i à l'isolant.
L_i est la longueur de la fibre moyenne de l'isolant.

Dans ce cas, on peut encore aboutir à une équation en différences finies explicites.

Il faut cependant noter que ces méthodes se heurtent généralement à la méconnaissance des valeurs suffisamment précises de la chaleur spécifique des isolants courants.

3. CALCUL AUTOMATIQUE DE LA RESISTANCE AU FEU DE L'OSSATURE.

Pour pouvoir utiliser les théories classiques de l'analyse limite pour le calcul de la résistance au feu du système, il est indispensable de conserver la notion de rotule plastique, donc d'adopter un diagramme contrainte-déformation élastique parfaitement plastique aux différentes températures (figure 3). La limite élastique R_e décroît alors avec la température suivant une loi connue.

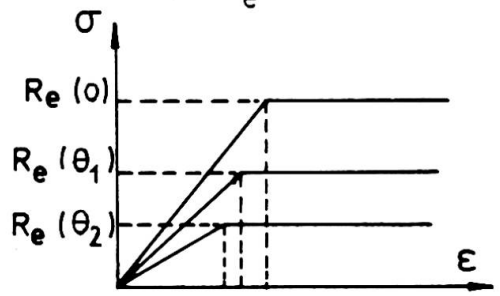

Figure 3

Il s'agit en réalité d'une approximation. Des résultats expérimentaux dus à Magnusson [7] ont en effet montré que le diagramme contrainte-déformation tend à devenir non linéaire aux hautes températures.

Une autre hypothèse concerne la répartition longitudinale de la température dans les profilés.

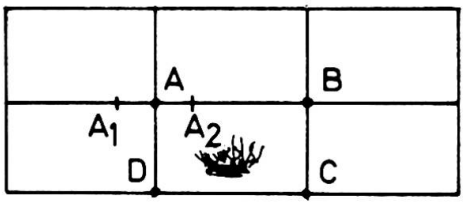

Figure 4

A la figure 4, on a représenté une maille incendiée dans une ossature (ABCD). Dans ce cas, on admet généralement que, pour chacun des éléments AB, BC, CD, DA se trouvant dans la zone incendiée, la température est uniforme d'un bout à l'autre. Aux extrémités A, B, C, D, la température varie brusquement lorsqu'on passe d'une zone incendiée à une zone non incendiée.

Les calculs que nous avons effectués montrent que la distance $A_1 A_2$ sur laquelle se produit le gradient thermique est généralement petite, ce qui justifie l'hypothèse admise.

Pour résoudre ce problème de manière automatique, il existe deux types de méthodes : les méthodes pas à pas et celles utilisant les techniques de la programmation mathématique. Nous avons donné la préférence aux méthodes pas à pas, qui paraissent mieux adaptées ici. En effet, le problème dépend de plusieurs paramètres ; pour le résoudre, il faut le décomposer en une succession de problèmes dépendant d'un seul paramètre.

La solution apportée est une solution du premier ordre, en ce sens que l'on n'a pas tenu compte des effets P.Δ. On a cependant prévu une vérification des colonnes au flambement.

La ruine détectée par le code de calcul correspond à l'apparition d'un des phénomènes suivants :

- formation d'un mécanisme d'ensemble ou local (mécanisme de poutre) ;
- instabilité d'une colonne comprimée et fléchie ;
- plastification par effort normal d'un élément massif.

En ce sens, le programme est entièrement automatique : il indique la formation successive des rotules plastiques, l'instant où ces rotules se forment et la durée de résistance au feu, correspondant à la formation d'un des mécanismes de ruine précités.

4. EXEMPLE D'APPLICATION.

Les considérations précédentes sont illustrées par le calcul de la résistance au feu d'un building de 3 étages. Les caractéristiques de l'ossature sont données à la figure 5.

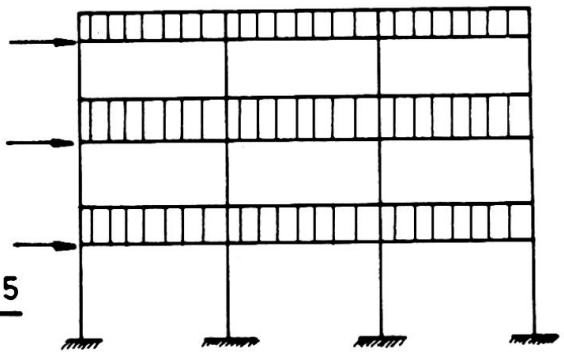

Figure 5

Protection : box - casing vermitecta 2cm

Charges	: vert.	hor.
toit	4 T/m	1 T
étages	5,8 T/m	2T

Deux cas différents d'incendies sont examinés. Dans le premier (cfr. figure 6), l'incendie se développe dans la cellule centrale inférieure. La rupture se produit par instabilité de la colonne indiquée après 1 h 23' d'incendie, alors que la température dans l'acier est de 475°. Dans le deuxième cas (cfr. figure 7), un premier foyer se déclare, puis 30 minutes plus tard, l'incendie se propage dans la cellule supérieure. Comme les éléments concernés sont beaucoup moins sollicités, la ruine n'apparaît qu'après 2 h 25', alors que la température dans l'acier est de 730°C.

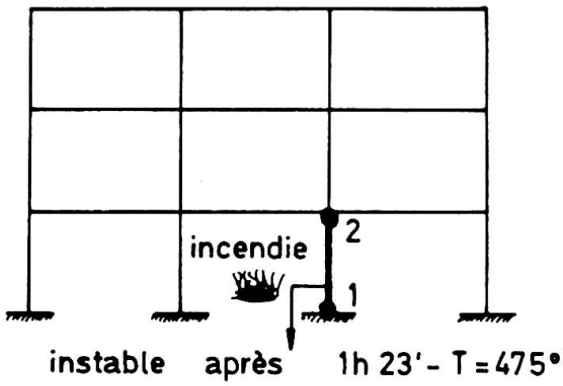

Figure 6

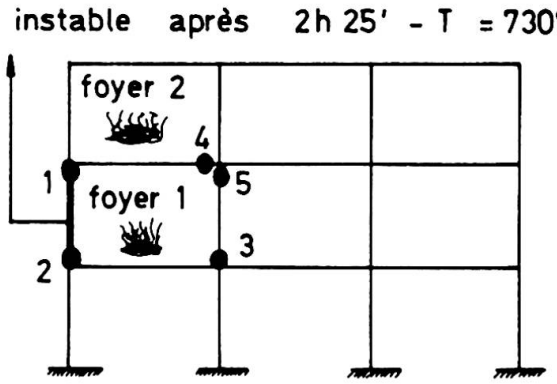

Figure 7

5. CONCLUSIONS.

Cet article fait apparaître l'intérêt des méthodes de simulation pour le calcul de la résistance au feu des structures. L'exemple précédent montre qu'on ne peut pas assimiler la résistance d'un élément essayé isolément à celle qu'il présente dans une ossature complexe. Il met en évidence l'importance de certains facteurs, comme le taux de travail des éléments et la localisation de l'incendie.

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RESUME

On présente une méthode théorique pour le calcul automatique de la résistance au feu des ossatures métalliques protégées et non protégées. Le procédé utilisé est apparenté aux méthodes de l'analyse limite des ossatures en acier doux, basées sur le concept de rotule plastique. Un exemple d'application fait apparaître que le choix d'une température critique unique n'est pas réaliste.

ZUSAMMENFASSUNG

Man stellt ein theoretisches Verfahren vor, um den Feuerwiderstand geschützter und ungeschützter Stahlbauten automatisch zu berechnen. Dieses Verfahren ist mit dem Traglastverfahren verwandt (plastische Gelenke). Ein Beispiel zeigt, dass die Wahl einer einzigen kritischen Temperatur unrealistisch ist.

SUMMARY

A theoretical method is presented for the automatic computation of the fire resistance of protected and unprotected steel structures. The procedure used is rather similar to the method of the limit analysis of steel structures, based on the concept of plastic hinge. An example shows that it is not realistic to adopt only one critical temperature.

Justification par le calcul du comportement au feu des structures métalliques

Feuerwiderstandsberechnung von Stahltragwerken

Fire Resistance Calculation of Steel Structures

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After having done in its research station a great number of fire tests sponsored by the Commission of European Committies (C.E.C.) and the European Conven tional Steelwork Associations (C.E.C.M.), the French Technical Center for Steel Construction (C.T.I.C.M.) has developed a method of fire resistance calculation of steel structures.

This calculation requires the knowledge of two different temperatures, which are :

1° The critical temperature of the structure (or only a part of it) which is the steel temperature when the structure can no longer hold its function, i.e. when it collapses either by loss of strength or because of a too large deflection.

2° The steel temperature at the end of the time of stability required by official fire safety rules.

Thus, the knowledge of these two temperatures gives us an answer to the question : will the structure collapse or will it not collapse during the required time of stability ? Or, in other terms : is the critical temperature of the structure higher or lower than the steel temperature at the end of the stability required time ?

Let us explain quickly these calculations :

The critical temperature is mainly dependent upon the loading and type of structure (i.e. statically or non-statically determinate structure) and end restraint conditions. It is not the same for all kinds of structures, as many people believe : it can be 700°C as well as 300°C !

Practically, its determination is made from a flow-chart relating to the temperature the loading rate, i.e. the quotient of the normal loading by the ultimate strength of the structure which is obtained by a plastic calculation. This chart is nothing but the curve giving the decrease with temperature of yield stress of steel (fig. 1).

The calculation is a little bit more complicated when the member is thermally restraint by the rest of the structure which is not affected by fire. In this case, one must calculate the structure rigidity with respect to restraint member in order to determinate the supplementary stress induced in the member. Some charts have been drawn to make this calculation easier (fig. 2).

The heating-up behaviour of non-protected steel member is directly read on a chart drawn from a heating formula supposing a heat transfer through the steel uniform an instantaneous (fig. 3). If the member is protected by direct application of fire resistant material such as vermiculite, plaster, etc.., the protection is introduced in the calculation by a single coefficient function of temperature which makes the summary of all the thermal properties of the protection in the normal conditions of use of the product. This coefficient is determinated by an heating test of protecting members. If there is moisture in the product, the method is slightly different to take into account the water vaporisation level. We have considered that spalling does not occur and that moisture is beneficial for fire endurance.

This method is always simplificated by charts giving directly for each product the necessary thickness of protection when the critical temperature and stability time are known (fig. 4).

In summary, this method, though it is justified by complex theoretical calculations, is quite easy to use because many charts have been established. It can be used whatever kind of fire is considerated (natural fire or ISO curve), but it cannot yet solve some particular disposals such as external columns, composite structures or members shelted by suspended ceilings or partition walls. Researches are in progress in France on all these points.

This method of calculation is exposed in detail in a document named "Verification par le calcul du comportement au feu des structures en acier" [1].

References

[1] "Verification par le calcul du comportement au feu des structures en acier" Projet de recommandations établies par le CTICM, avril 1976

FIGURES

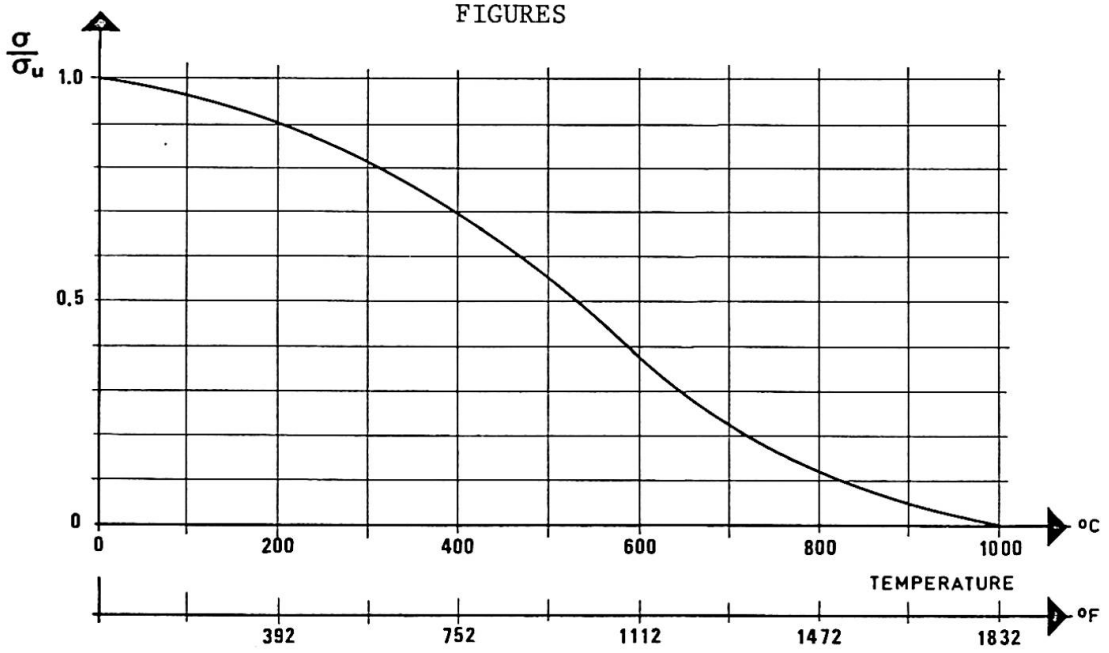


Figure 1 : Chart for calculation of critical temperature.
σ : normal loading
σ_u : ultimate strength

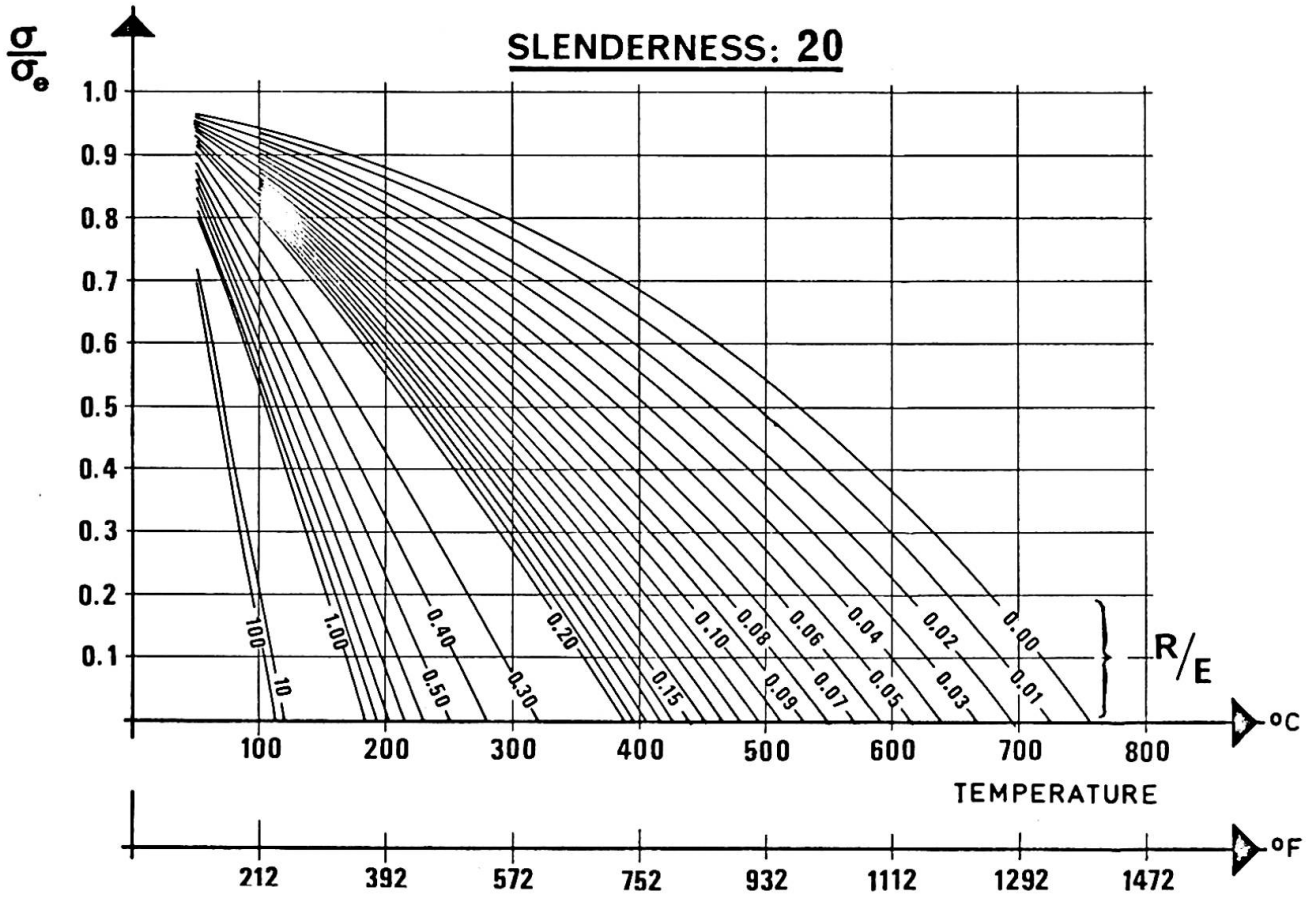


Figure 2 : Example of chart for calculation of critical temperature of a thermally restraint member.
σ_e : yield stress
R/E : structure rigidity with respect to restraint member

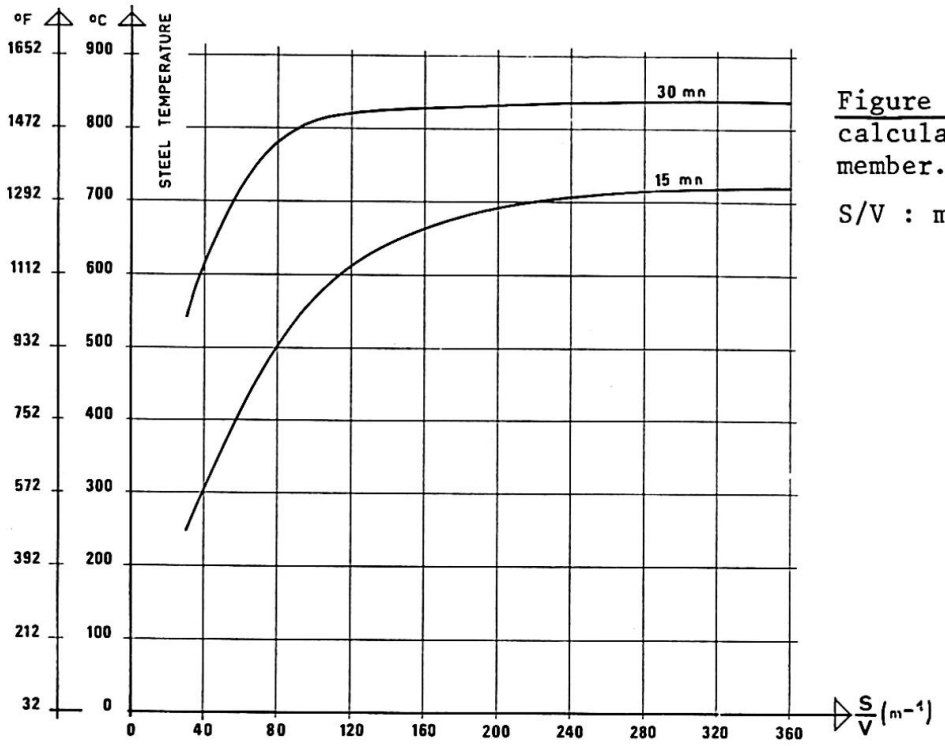
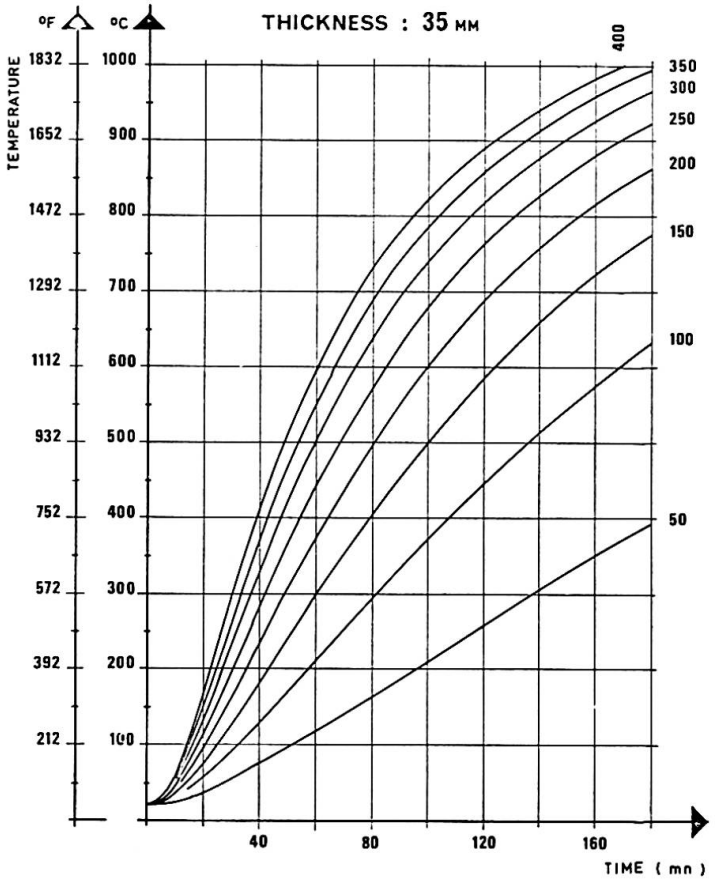


Figure 3 : Chart for heating calculation of non-protected member.

S/V : massivity of member

Figure 4 : Example of chart for calculation of thickness of a given product of protection.



Theoretical and Experimental Analysis of Steel Structures at Elevated Temperatures

Analyse théorique et expérimentale des constructions métalliques soumises à des températures élevées

Theoretische und experimentelle Untersuchung von Stahlkonstruktionen bei hohen Temperaturen

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1. INTRODUCTION

In principle the reduction in bearing capacity of steel structures under fire action can be determined on the basis of the mechanical properties at elevated temperatures. Some authors [1, 2] solve the problem by using data derived from standard material tests at various temperatures, applying conventional time independent methods of analysis. This procedure has the adventage of being ready for use to designers, but the material tests rather arbitrarely include the creep behaviour, which becomes manifest at temperatures over 400°C. Other authors like Thor [3] and Eggwertz [4] use data derived from conventional creep tests, which are fed into computer programmes, providing the time dependent creep behaviour of structural elements. This method is more reliable but is rather complicated and so far only solutions have been obtained for simple structural elements like beams [3] and columns [4]. Extending this method to more complicated structures like frames encounters practical difficulties. So far, both methods could only be checked by fragmentary experimental data.

In order to arrive at consistent and practical solutions some years ago in the Netherlands work was started combining a theoretical approach with an, in this field, new experimental technique of small scale model tests on beams, columns and frames. For the models, steel bars were used with rectangular cross-sections, with dimensions varying between 5 and 20 mm. The length of the columns and the span of beams varied between 100 and 400 mm. This makes possible a uniform temperature distribution accross and along the members, an assumption which also applies to the theoretical analysis. This contribution summarizes the results obtained so far; more detailed information is published in [5, 6].

2. DEFORMATION CHARACTERISTICS OF STEEL AT ELEVATED TEMPERATURES

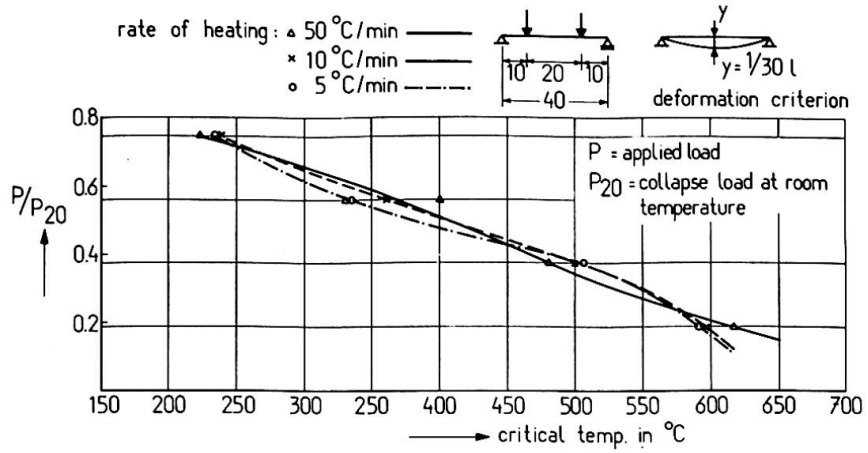
In case of fire in general a structural member will be under constant load and subjected to a temperature increase as a function of time. Depending on the severety of the fire and the insulation of the structural member, the rate of heating can vary. The influence of the rate of heating on the deformation behaviour was studied by analyzing the behaviour of beams and columns on model

scale, with a constant load and heated with different heating rates. Linear heating rates were chosen of 50°C per minute (approximately corresponding to an unprotected steel member); 10°C per minute (normally protected steel member) and 5°C per minute (heavily protected steel member). In figure 1 the results of beams with a span of 400 mm and a cross-section of 6x6 mm^2 are summarized. On the vertical axis the applied load is plotted, as a fraction of the collapse load at room temperature, determined experimentally. On the horizontal axis the critical temperature is plotted, being the temperature at which the deflection becomes 1/30th of the span. It follows from the tests that the heating rate does not influence the deformation behaviour in a significant way. For columns similar results were obtained [5]. The results imply that the collapse temperature of steel elements can be considered as time independent, and are consequently not influenced by the heating history. This conclusion makes a theoretical approach possible, which is identical to well known methods at room temperature. Instead of one stress-strain diagram a family of diagrams for different temperatures should than be used. In those stress-strain diagrams creep can be considered as incorporated in the relationships.

To find these relationships non-conventional warm-creep tests were carried out with a loading- and heating procedure similar to the tests on beams described before. A standard cylindrical testpiece was subjected to a constant load and an increasing temperature. In figure 2 a typical result is given. It can be seen that at a certain temperature the elongation increases at constant temperature. This phenomenon is called thermal activated flow. After a certain elongation strainhardening occurs. With Harmathy's theory, slightly modified, the influence of the rate of heating could be determined theoretically. As can be seen the influence of the rate of heating is quite modest. From the warm-creep tests the stress-strain relationships can be derived with a simple transformation. It is emphasised that this transformation is only justified for "practical" heating rates (i.e. between 5 and 50°C per minute and temperatures not over, say 600°C). In addition also stress-strain relationships were obtained by analyzing the small scale beam tests. It appeared that the results obtained by these tests were in reasonable agreement with those obtained by warm-creep tests. In figure 3 a family of stress-strain relationships is presented. The phenomenon of thermal activated flow is visable in this figure by the gap between the relationships applying to 200°C and 300°C. In the subsequent discussion we will consider the family of stress-strain diagrams at elevated temperatures as a starting point for the theoretical analysis of beams, columns and frames.

3. RESULTS ON BEAMS

In figure 4 a typical result is given of a simple supported beam loaded with two point loads. As can be seen the results of the calculations are in good agreement with the tests. In this case the complete deformation process as a function of the temperature is given. In practice in many cases only the collapse temperature under a given load is of interest. A much less elaborative procedure can than be used by applying simple plastic theory. In that case only the yield stress at a given temperature derived from the stress-strain relationships has to be used. In figure 5 results are shown of tests on beams with variable end restraints. The beams are framed into columns of which buckling was prevented. By varying the plastic moments of the beams and the columns, the restraint conditions can be varied. It can be seen that the theory is in good agreement with the test results.



temperature at which a maximum midspan deflection is reached as function of the load and the rate of heating (FeE 24)

fig. 1

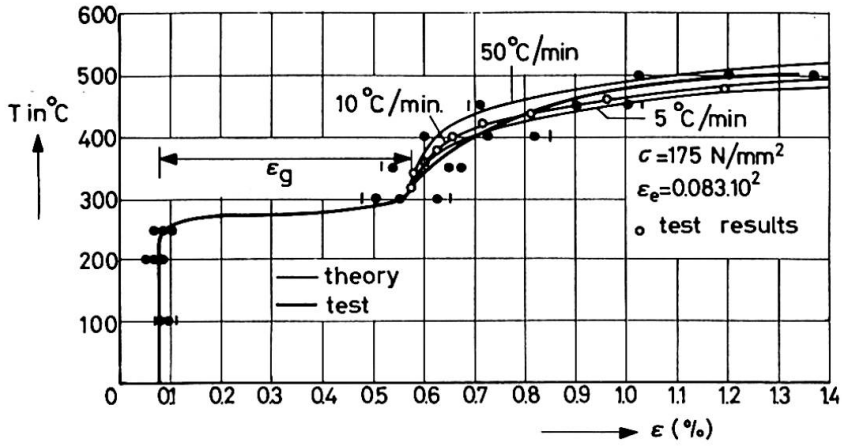
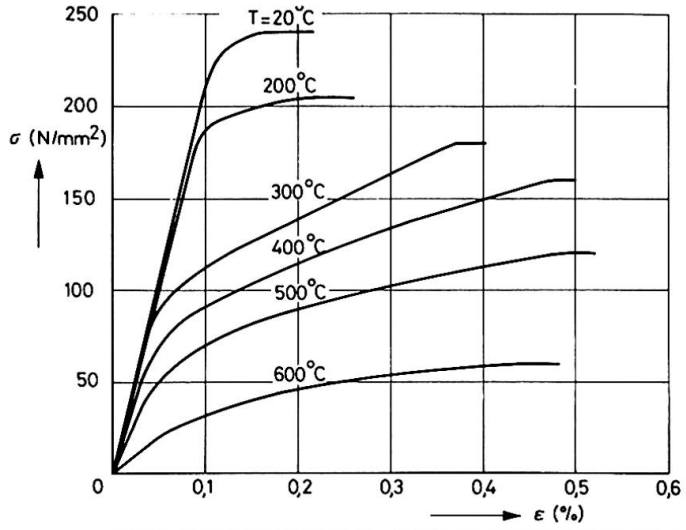


fig. 2 T- ε relations for different heating rates



stress-strain relationship of FeE 24 at elevated temperatures

fig. 3

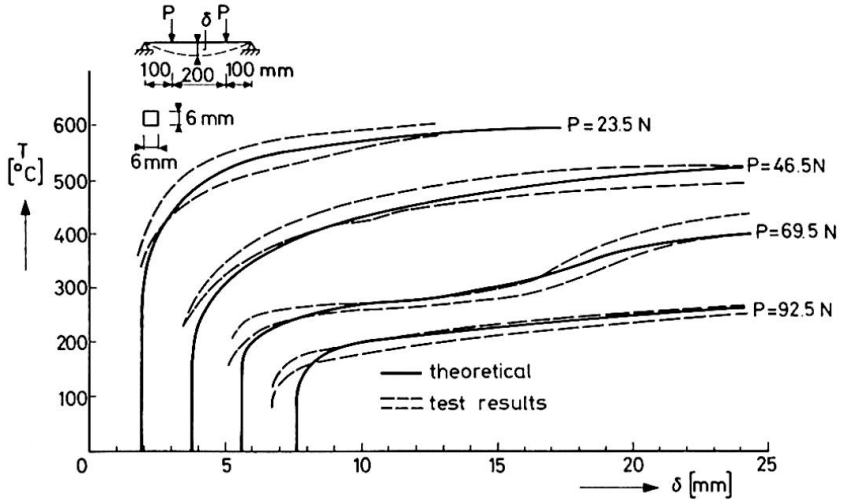


fig. 4 temperature- deflection curves of beam at certain loadlevels.

4. RESULTS ON INDIVIDUAL COLUMNS

In figure 6 computated buckling curves are presented together with experimental results from small scale models. The computations are based on the assumption of an initial out-of straightness of the columns. The value of the out-of straightness is chosen in such a way that at room temperature the computated value of the load bearing capacity coincides with experimental values. In the same figure the buckling curve based on the Dutch design code is given.It is pointed out that the result established for 600°C coincides with a creep buckling analyses by Eggwertz [4] of a column with a slenderness ratio of 45 subjected to a temperature-time history with a maximum of 600°C (allowance is made for difference in yield strength for the respective materials).

5. RESULTS ON BRACED AND UNBRACED FRAMES

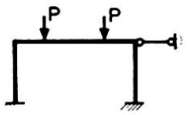
The analysis can take place in the same way as applicable to structures at room temperature, and needs no extensive discussion. As can be seen from figure 3 the deformation behaviour is non-linear, and as a consequence for the structural analysis, computer techniques must be used. Recently, however, computer programmes are available in which physical non-linearity as well as geometrical non-linearity can be taken into account. In connection with physical non-linearity there is one complication to be discussed here. The unloading characteristics of steel at elevated temperatures are not well established yet. For the time being, an elastic unloading behaviour is assumed, of which the path of unloading coincides with the path of loading. This assumption in general leads to lower bound solutions.

In figure 7 the results of theoretical and experimental analyses are summarized for braced frames. By variation of the cross-sectional dimensions two types of frames were obtained with different effective slenderness ratio's. Two different load levels P were applied, defined as a fraction of the collapse load P_{20} at room temperature. In addition loads K have been applied, chosen in such a way that the linear elastic moment at the top of the column equals one half of the moment producing the yield stress in the outmost fibres of the column. It follows from figure 7 that the theoretical predicted collapse temperatures are slightly lower than those resulting from the experiments. This trend is not surprising considering the simplifying assumption of elastic unloading characteristics of steel at elevated temperatures.

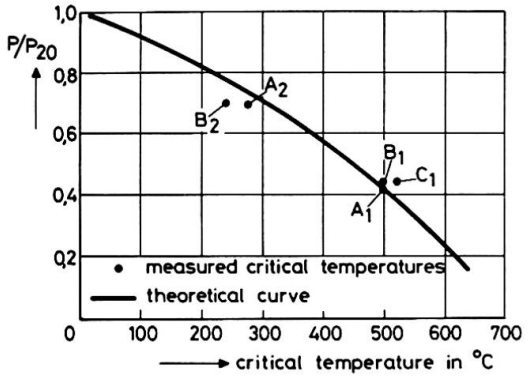
Analyzing the stability of unbraced frames it becomes apparent that the initial inclination of the frame under vertical load is an important factor. This is illustrated in figure 8 where a typical result is given of an unbraced frame with the same loading type as for the braced frames. It follows that the theoretically derived collapse temperature is substantially influenced by the mode and magnitude of the inclination. As can be expected maximum values are found if the mode of inclination is symmetric. It can be seen that the test results are conservative compared with the theoretical values. In [6] results are described of unbraced frames with additional horizontal loads.

6. CONCLUSIONS

Summarizing it can be stated that the presented method, describing the behaviour of beams, columns and frames at elevated temperatures seems reliable. A basic feature is that the deformation characteristics of steel at elevated temperatures can be considered as time independent. Further activities are going on to impliment the method for real structures and make it ready for use

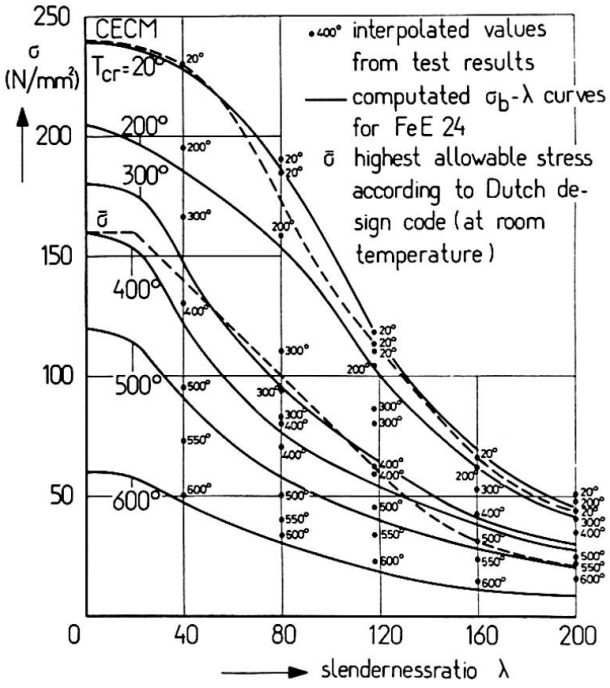


P = applied load
P_{20} = collapse load at room temperature



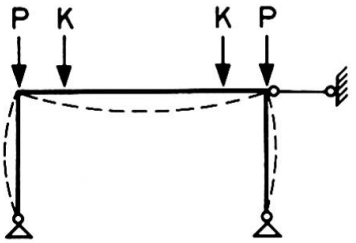
comparison between calculated and measured critical temperatures of beams with different end restraint (instability prevented)

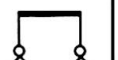
fig. 5



relationship between critical stress and slenderness-ratio at different temperatures

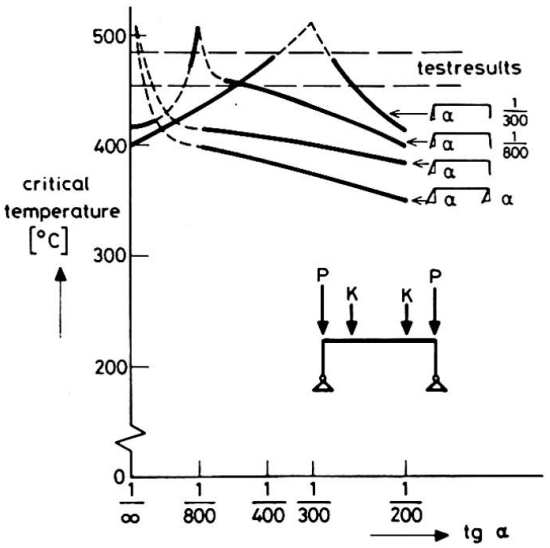
fig. 6



	P/P20	Tcrit tests	Tcrit analysis
	0,4	525	500
	0,6	380	350
	0,4	590	515
	0,6	530	425

theoretical and test results of braced frames

fig. 7



relationship between the inclination α and the critical temperature of an unbraced frame at elevated temperatures ($P = 0{,}6\ P_{20}$)

fig. 8

to designers. The assumption of a uniform temperature distribution along and accross the members is not essential. An extension to non-uniform heating can easily be realized.

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SUMMARY

In this study the deformation characteristics of steel at elevated temperatures are derived from creep tests as well as from small scale model tests on beams. As a result the load bearing capacity of steel structures under fire action can be determined with the well known methods used at room temperature. Both theoretical and experimental results of beams, columns and frames are summarized.

RESUME

Dans cette étude les caractéristiques de déformation de l'acier aux températures élevées sont déterminées par des essais de fluage et par des essais à échelle réduite sur des poutres. La résistance des constructions métalliques au feu peut ainsi être déterminée à l'aide des méthodes bien connues, utilisées aux températures ordinaires. Les résultats théoriques et expérimentaux sont présentés pour des poutres, poteaux et cadres.

ZUSAMMENFASSUNG

In dieser Untersuchung sind die Verformungseigenschaften von Stahl bei hohen Temperaturen aus Kriechversuchen und aus Modellversuchen in kleinem Massstab an Trägern ermittelt. Die Tragkraftfähigkeit von Stahlkonstruktionen unter einer Brandbeanspruchung kann mit den bekannten Verfahren bei normalen Temperaturen ermittelt werden. Theoretische und experimentelle Resultate für Träger, Stützen und Rahmen sind zusammengefasst.

Creep Buckling of a Steel Column in a Temperature-Time History Simulating a Fire

Flambage par fluage d'un poteau en acier selon un diagramme température-temps simulant un incendie

Kriechknicken von Stahlstützen in einem Temperatur-Zeit Verlauf, der einen Brandfall simuliert

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1. INTRODUCTION

At temperatures exceeding 500°C ordinary carbon steels show a high creep rate even at low tensile or compressive stresses. In a paper published in the Preliminary Report of the 10th Congress of IABSE [1] the author presented results of creep buckling calculations for a steel column of section HE 240B with a yield strength of 300 MPa. A computer programme prepared at the Aeronautical Research Institute of Sweden [2] was used to obtain the creep buckling life of columns of various lengths subjected to a number of different mean compression stresses assuming a constant temperature of 600°C. A diagram was given where the buckling stress was plotted versus the slenderness ratio λ for times of exposure to this temperature ranging from 0.2 to 2.0 h. The diagram showed quite clearly that time is an important parameter which must be considered when analysing the buckling strength of a steel column at elevated temperatures.

The temperature during a fire varies, however, with a rise from room temperature to a maximum, determined by the fire load, after which the cooling starts, usually at a much lower rate than the heating. The computer programme mentioned was therefore modified to allow variations in temperature. A few calculations with such temperature histories assuming maximum temperatures 600° and 650°C respectively were carried out and presented already in Ref [1]. This investigation has been continued. By repeated life calculations, the compression stress was determined which just resulted in creep buckling collapse at the end of a temperature-time history with a maximum of 600°C. This was done for the slenderness ratios λ = 30, 45, 60 and 90.

2. CALCULATIONS

The modified version of the creep buckling computer programme is able to take into account a linear increase in temperature from an initial value to a maximum ϑ_{smax}, then a constant period and finally a linear decrease in temperature. The creep law of Norton-Odqvist is utilized

$$\dot{\varepsilon} = k\sigma^n$$

where the exponent $n = 4.9$ is assumed to be constant for all temperatures considered, while the creep coefficient k varies with the temperature T°K

$$k = 1.88 \times 10^{-11} \exp (44.7 - 39000/T)$$

Young's modulus, including elastic and plastic deformation, as well as primary creep, is determined by the relation

$$E_o = 325000 - 404\ \vartheta_s \text{ MPa}$$

The numerical calculations were performed for a temperature-time history shown in Fig 1 with heating and cooling rates 1000°C/h and 333°C/h respectively, and further $\vartheta_{smax} = 600°C$ lasting for a period of 0.2 h. The radius of gyration in the buckling direction of the chosen column section HE 240B is i = 0.10 m. The assumed lengths were 3.0, 4.5, 6.0 and 9.0 m.

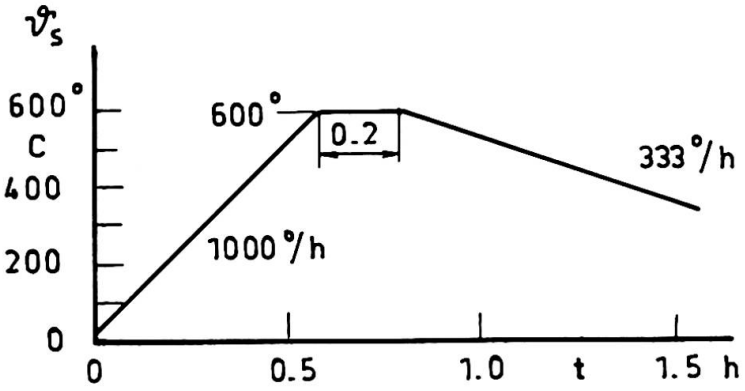

Fig 1 Temperature- time history introduced into computer programme

3. RESULTS AND DISCUSSION

After a number of trials with different loads on the column section it was found that for the slenderness ratios $\lambda = 30$, 45, 60 and 90 the mean stresses $\sigma = 60$, 51, 41 and 28 MPa respectively would result in creep collapse just at the moment of the temperature-time history during the cooling phase where creep had practically ceased. The results have been plotted in Fig 2 together with the creep buckling curves obtained earlier [1] for constant temperature exposure times. It was discovered that the value determined before with $\lambda = 45$, for a history with the maximum temperature 600°C, was somewhat too high due to an unconservative interpolation. The temperature-time history of Fig 1 seems to have about the same effect as if the column had been exposed to a constant temperature of 600°C during 0.5 h. This result is not surprising since the temperature in Fig 1 exceeds 500°C, where the rapid creep starts, during 0.6 h. Further investigations are likely to show the feasibility

of using constant temperature buckling curves to determine the critical stress of a column subjected to a fire temperature history.

The buckling analysis presented in Fig 2, although considering a realistic fire history of the mean temperature of the steel column, still neglects other complications met in practice such as temperature gradients and residual stresses within the column, as well as restraining forces from other structural elements. Thus, it is not claimed that Fig 2 may be used directly for design purposes. It seems to represent a substantial improvement, however, in comparison with the two dotted curves given by the general reporters, Theme III, in the Introductory Report of the Congress [3]. The analysis clearly shows that creep has to be taken into account at fire temperatures exceeding 500°C, where time is consequently an important parameter. This conclusion was supported in the Prepared Discussion contribution by Fukumura [4].

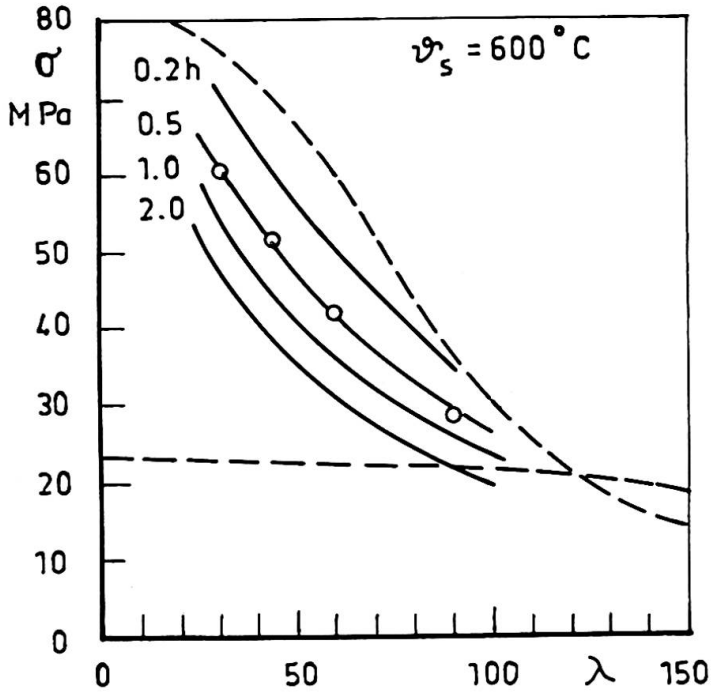

Fig 2 Buckling stress versus slenderness ratio for various times of exposure to 600°C and for temperature-time history with maximum temperature 600°C

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SUMMARY

The creep buckling stress was determined by computer analysis for steel columns subjected to a realistic fire temperature history with a maximum of 600 °C. The results were compared to earlier calculations with constant temperature of 600 °C. The influence of creep was still found to be important.

RESUME

La contrainte de flambage par fluage de poteaux en acier a été déterminée par des calculs numériques selon un diagramme température-temps simulant un incendie avec un maximum de 600 °C. Les résultats sont comparés aux calculs déjà faits pour une température constante de 600 °C. On constate que l'influence du fluage reste importante.

ZUSAMMENFASSUNG

Die Kriechknickspannung wurde für Stahlstützen durch ein numerisches Programm für einen Temperatur-Zeit Verlauf mit maximum 600 °C bestimmt, der einen Brandfall simuliert. Das Resultat wird mit früheren Berechnungen mit einer konstanten Temperatur von 600 °C verglichen. Der Einfluss des Kriechens ist von Bedeutung.

Inelastic Behaviour of Protected Steel Columns in Fire

Comportement inélastique des poteaux protégés contre l'incendie

Nichtelastisches Verhalten von feuergeschützten Stahlstützen

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1. Introduction

The fire resistance of structural members has been determined in the past by subjecting isolated members to standard fire tests. More recently, calculations based on heat transfer and the structural properties of materials at high temperature have been used to replace the costly and time-consuming standard fire tests. In the past the cost of testing precluded considerations of assuming more realistic situations corresponding to columns in continuous structures subjected to fires representing actual conditions. These more realistic situations can now be studied by computer calculation.

In this work three aspects for protected steel columns are studied: (1) the stress distribution in a column during fire, (2) the interaction of an interior column exposed to fire with the surrounding building structures, and (3) the fire resistance of columns which take into account creep deformation behavior.

This work is a continuation of Ref.1 and the method of analysis regarding determination of stress, strain and deformation of the column are contained therein.

2. Calculation of Temperature Rise

A computer program was written to calculate the temperatures with in a column when the temperature outside four faces was raised in the manner specified for fire tests in JIS A 1304.

Most columns used in buildings have sections less than 15 cm thick. In such cases detailed calculation of temperature distribution in the steel cross section is unnecessary, and one-dimentional model of the heated column can be usefully employed.[2] The model consists of a steel plate having the same cross-sectional and surface areas per unit height as the four sides of the heated column, with the edges and unexposed side perfectly insulated. This model permits use of a one-dimentional numerical procedure.

In calculating the temperatures, furthermore, it was assumed that fire protection panels became divided into a dry zone and a wet zone, within each of which the thermal properties were taken to be constant.

It was also assumed that vaporization occured at atmospheric pressure at the interface between these zones, and that therefore interface moved away from the heated face as drying penetrated into the protection. Typical results, for the temperatures in H cross-section steel columns with 3 or 5 cm thick protection, are shown in Figures 1 and 2.

3. Stresses and Strains in a Column during Fire

To analyse the strain and stress in a column section, the following assumptions were made:

(1) Plane sections remain plane. This assumption is approximately correct for long prismatic members in continuous construction.

(2) The free expansion of steel due to temperature change, ε_T, is assumed as follows (see Fig.3) :

$$\varepsilon_T = 5.04 \times 10^{-9} T^2 + 1.13 \times 10^{-5} T \quad \cdots\cdots\cdots (1)$$

where T is in degrees Centigrade.

(3) The relationship between steel stress σ, and strain ε, in compression, is assumed as follows (see Fig.4) :

$$\varepsilon E_0(T) = C(T)\sigma - [1 - C(T)]\sigma_y(T) \ln[1 - \sigma/\sigma_y(T)] \quad \cdots\cdots (2)$$

where

$$E_0(T) = 2.15 \times 10^6 [0.745 + 0.137 \ln[-0.01T + 6.3]] \quad (kg/cm^2)$$

$$\sigma_y(T) = 5.0 \times 10^3 [-4.49 \times 10^{-9} T^3 + 3.57 \times 10^{-6} T^2 - 1.41 \times 10^{-3} T + 1.027] \quad (kg/cm^2)$$

$$C(T) = -4.0 \times 10^{-4} T + 1.0$$

In this expression $\sigma_y(T)$ is the steel yield point (SM 58), $E_0(T)$ is the initial tangent modulus and C(T) is a parameter that affects the form of the stress-strain curve as shown in Figure 4, in which T is expressed in degrees Centigrade. Besides these data, the material description must include behavior during unloading and in tension. This is arranged by making the following two assumptions: (1) Behavior in tension is the same as that in compression, and (2) Behavior during unloading from (or reloading to) a previously obtained value of a compressive stress given by equation 2 is linear $d\sigma/d\varepsilon = E_0(T)$, the initial tangent modulus.

(4) To evaluate the creep deformation, it is assumed that the creep strain for SM 58 is related to time t, absolute temperature T', and current stress σ in the form such that (see Fig.5)

$$\varepsilon_c = [10^{c - a/2.3T'}] \times [2.37\sigma/5.0]^{b/2.3T' + \alpha}] \, t^n/n \quad \cdots\cdots (3)$$

where $c = 20.53$ $b = 1.9 \times 10^4$ $\alpha = -7.25$ $n = 0.35$ $a = 4.5 \times 10^4$

t: minutes σ: kg/mm^2 ε_c: %

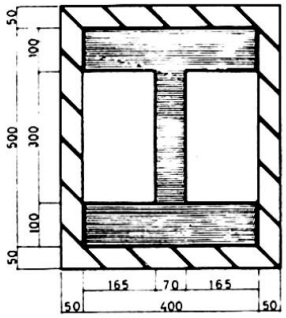
The strain-hardening creep low is applied for the calculation of the nonstationary creep deformation.

(5) The column is divided into blocks with 1/20 or 1/40 column length in the axial direction and each block is subdivided further into 20 elements in the cross sectional direction. For each element the strain, stress and material properties are assumed uniform.

(6) Initial stresses and strains in the cross-section before the fire, are determined, assuming that the axis of the column has initially the form of a sine curve which takes into account the various imperfections in a column.

(7) It is assumed that there are no cases in which a steel column subjected to compression will buckle torsionally or locally and the effect of shearing force on the deformation can be neglected. Buckling of the column will occur in the plane of minimum flexural rigidity.

Figure 1 SECTION



STEEL COLUMN
(SM58)

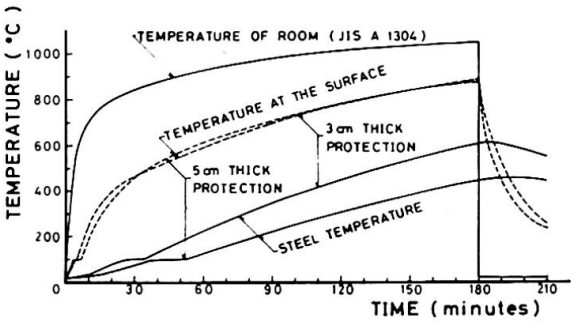
Young's modulus	2.15×10^{6} Kg/cm²
Strain corresponding to the yield stress	2.50×10^{-4} %
Yield stress	5.00×10^{3} Kg/cm²

(20°C)

FIRE PROTECTION
(CONCRETE)

Thickness	3.0 or 5.0 cm
Percentage of moisture content	4.0 %
Specific gravity	2340 Kg/m³

Figure 2 CALCULATED TEMPERATURE OF PROTECTED STEEL



FREE THERMAL STRAIN-TEMPERATURE CURVE

Figure 3

$$\varepsilon_T = 5.04\cdot10^{-9}\theta^2 + 1.13\cdot10^{-5}\theta$$

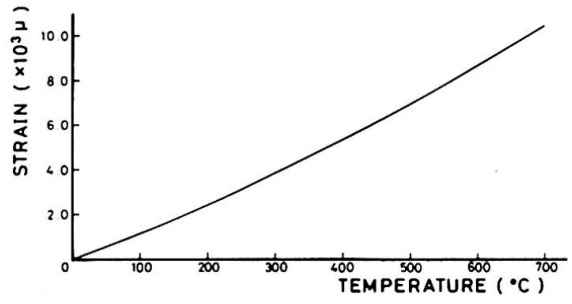


Figure 4 ASSUMED STRESS-STRAIN CURVE
(based on Harada and Furumura's Data)

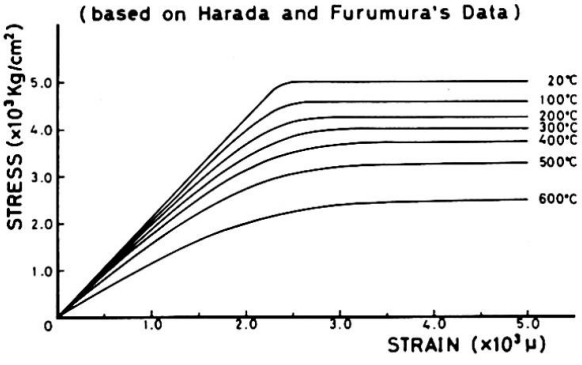
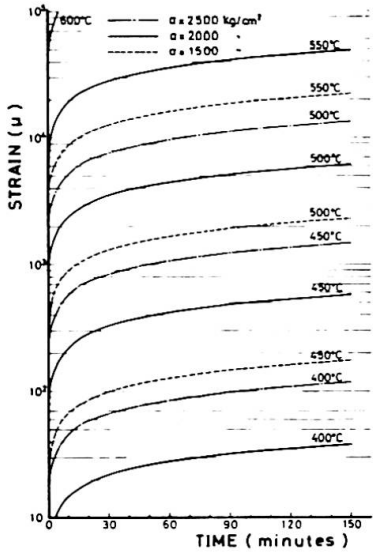
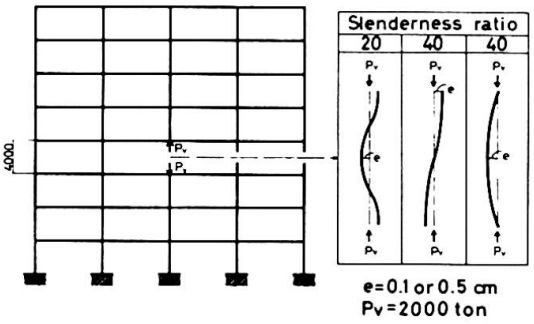


Figure 5 NONSTATIONARY CREEP



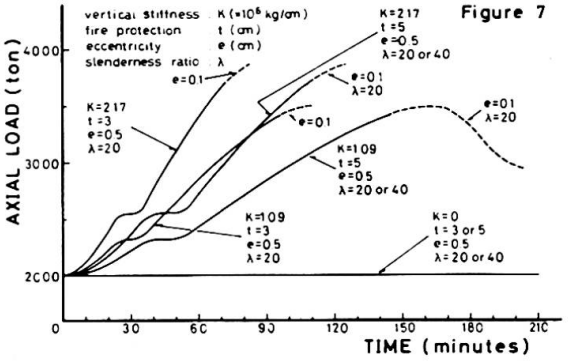
STRUCTURAL ARRANGEMENT OF BUILDING FRAME AND HEATED COLUMN

Figure 6



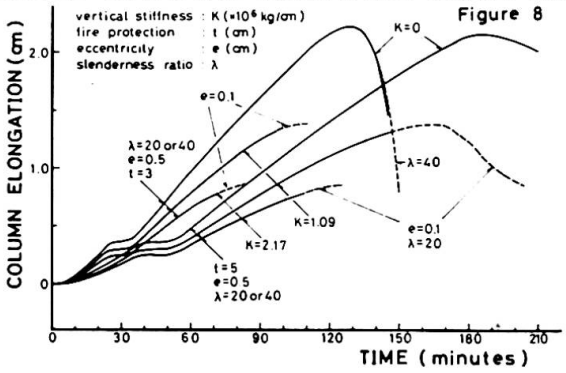
COLUMN-STRUCTURE INTERACTION DURING FIRE

Figure 7



COLUMN-STRUCTURE INTERACTION DURING FIRE

Figure 8



Based on these assumptions, the distribution of strain and stress is calculated for each 5-minute interval during fire. The method of analysis is to determine, by trial and error, a linear strain distribution in the steel which provides equilibrium of the cross-section.

4. Interaction of a Single Column and Surrounding Structure

A column expands during a fire and, because of the resistance of the surrounding structure to this expansion, more than the initial dead plus live load is attracted to the column.

If every column in a floor were subjected to the same fire, all the columns would expand equally and additional load would not be attracted to any one column. If only one of the floor columns were exposed to fire, the expansion of the column would be resisted by the surrounding structure. The greater the number of slabs above the column, the greater the resistance, and the greater the load attracted to the column.

The correct solution to the interaction problem requires a trial and error determination of moments, curvatures and displacements along the column for each point in time.

5. Results of Calculation

It is clear that a combination of axial load and axial and flexural restraint would arise with the structural arrangement shown in Figure 6.

To calculate lateral deflection, two cases of the flexural restraint for H cross-section steel column are assumed: (1) The column has both ends built in, and (2) The column is built in at the lower end and is free to displace laterally at the upper end but is guided in such a manner that the tangent to the deflection curve remains vertical, and that is equivalent to the case of a column with hinged ends.

The former assumption is considerably in error early in a fire during elastic conditions, but appears to be approximately correct as the column approaches failure when it bends with large inelastic strains at the critical sections. The latter is considered to be corresponded to the case in which every column in a floor would be subjected to the same fire.

Figure 7 shows how the applied load (initial load= 2000 ton/a column) increases by the interaction effect. Figures 8 and 9 to 11 show the corresponding column lengthening and lateral deflection, respectively.

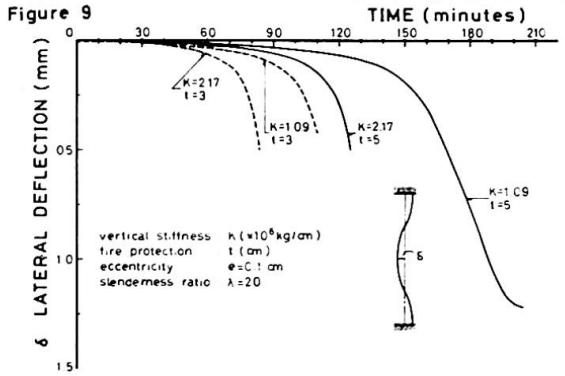
The interaction effect is shown clearly in Fig.7. The curve labelled "K=0", which corresponds to the assumption of no ineraction, serves as a basis for comparison. As the vertical stiffness of the surrounding structure increases from 0 to 1.09×10^6 kg/cm, which corresponds to 1/5 vertical stiffness of a column with 4 m length, the applied load more increases and the fire resistance is decreased, as seen by comparing column movements. As the vertical stiffness is increased further to 2.17×10^6 kg/cm, the fire resistance is more decreased. The fire resistance is also effected by the protection thickness.

Figures 12 to 14 show the variations of the strain, stress and tangential modulus in the extreme element on the convex side of the critical section. Figure 15 and 16 show the variations of the curvature distribution of the columns.

From the numerical results, the column is not greatly affected by creep until the steel temperature reaches 450°C. After 450°C,

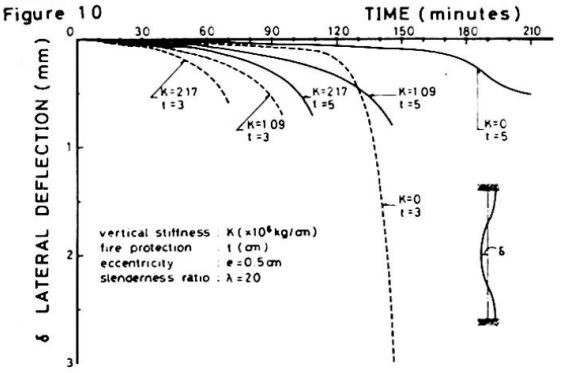
COLUMN-STRUCTURE INTERACTION DURING FIRE

Figure 9



COLUMN-STRUCTURE INTERACTION DURING FIRE

Figure 10



COLUMN-STRUCTURE INTERACTION DURING FIRE

Figure 11

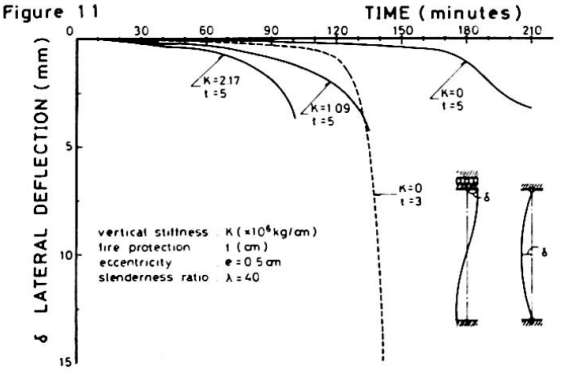


Figure 12 STRAIN-TIME CURVES DURING FIRE (due to stress)

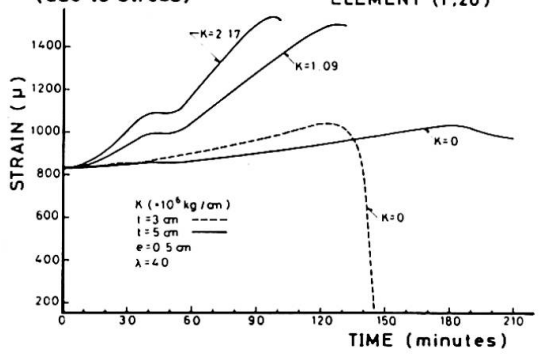


Figure 13 STRESS-TIME CURVES DURING FIRE

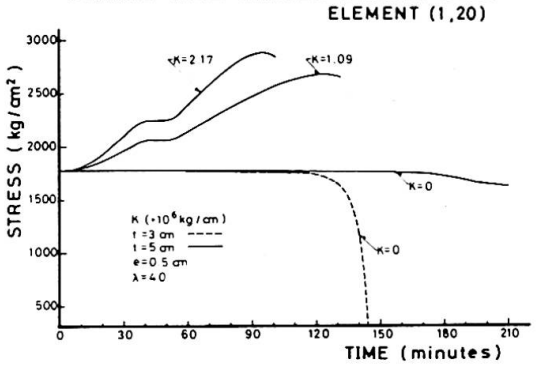
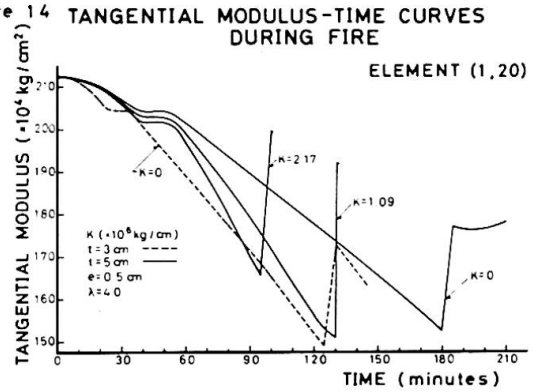
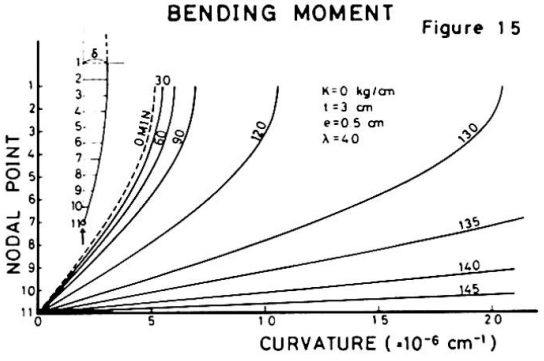


Figure 14 TANGENTIAL MODULUS-TIME CURVES DURING FIRE



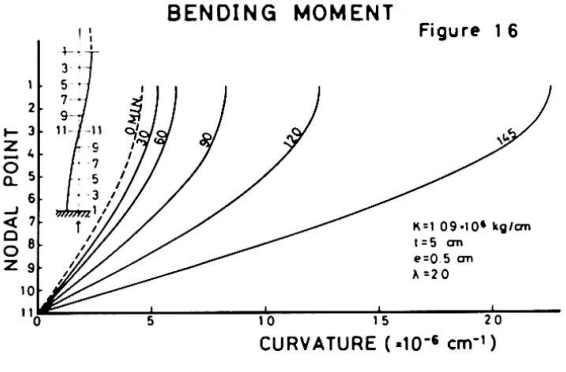
CURVATURE DIAGRAMS due to BENDING MOMENT

Figure 15



CURVATURE DIAGRAMS due to BENDING MOMENT

Figure 16



the creep rate increases rapidly and the column causes chord shortening, followed by lateral bending and relief of the applied load.

For very stiff vertical resistance the column buckles quite early, and the greater the initial deflection, the earlier the buckling of the column.

6. Conclusions

This study represents an initial attempt to evaluate in detail the behavior of a steel frame structure in a fire.

The creep deformation of columns in transient heating processes is of considerable interest to engineers working in the field of fire protection.

A numerical technique has been described in Ref.(1). The technique utilized a creep model was originally suggested by England[3] and Dougill[4] and expanded by the authers to suit certain practical requirements. The calculations that have been undertaken have been based on an idealized model of material behavior and it is doubtful whether the numerical values obtained have any significance in themselves. However, some aspects being important to understand modes of behavior of columns in fire have been found out and the information obtained from such detailed analysis is very useful in the future development of a simple and rational procedure for fire safety design of high rise steel structures.

7. References

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SUMMARY

The study shows the inelastic behaviour of protected steel columns under fire action. It is shown that the effect of creep on the buckling strength is very important at temperatures more than 450° C.

RESUME

L'étude s'occupe du comportement inélastique des poteaux métalliques protégés contre l'incendie. Il est montré que l'influence du fluage est très importante pour les températures supérieures à 450° C.

ZUSAMMENFASSUNG

Das nichtelastische Verhalten von geschützten Stahlstützen bei Brandbeanspruchung wird untersucht. Es wird gezeigt, dass dem Kriechen bei Temperaturen über 450° C grosse Bedeutung zukommt.

Design of Fire- and Impact-Resistant Ceilings in a Medieval Castle

Conception de plafonds résistant aux chocs et à l'incendie, dans un château médiéval

Entwurf von stoss- und brandsicheren Decken in einem mittelalterlichen Schloss

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1. Introduction

Karlštejn Castle is a quite exceptional example not only among Czech castles but in the whole of Middle Europe at all. It was built in the 14th century by Charles IVth, Czech king and since the year 1346 the Emperor of the Roman - German Empire, one of the greatest rulers of his time. The Castle /Fig. 1/ was erected to be not only a place of private retreat for the monarch, but also a noble shrine for a large collection of relics of saints which was enlarged in the year 1350 with crown - jewels of the Roman - German Empire. The most costly part of the treasure was, however, the crown of Charles the Great, a symbol of sovereign power over the whole Middle Europe.

Fig. 1 Karlštejn Castle

Important parts of the Castle are the Emperor's residence adjoined directly with the neighbouring Chapel of Virgin Mary by means of a wooden passage.

This church in the second floor of the tower - building was covered with an ornamental beam - ceiling. On three walls mural paintings from the Apokalypse are preserved together with three other scenes concerning directly important events leading to the foundation of the Castle. They are of extraordinary importance in the Gothic painting being ones of the first realistic portraits, especially those of Charles IVth. The Church of Virgin Mary had a direct connection with a donjon, which was the most solid and most important building of the whole Castle. The central room of this donjon and at the same time of the whole Castle is the Chapel of the Holy Cross on the second floor, characterised by Gothic windows, glassed partly with finely cut semiprecious stones. In a special niche behind the altar a treasure of greatest importance for the State - the crown jewels of the Roman - German Empire - was hidden. However, the greatest treasure for us now is a large medieval set of 128 Gothic tablets, representing saints and saintesses in half - figures in rare completeness /Fig. 2/.

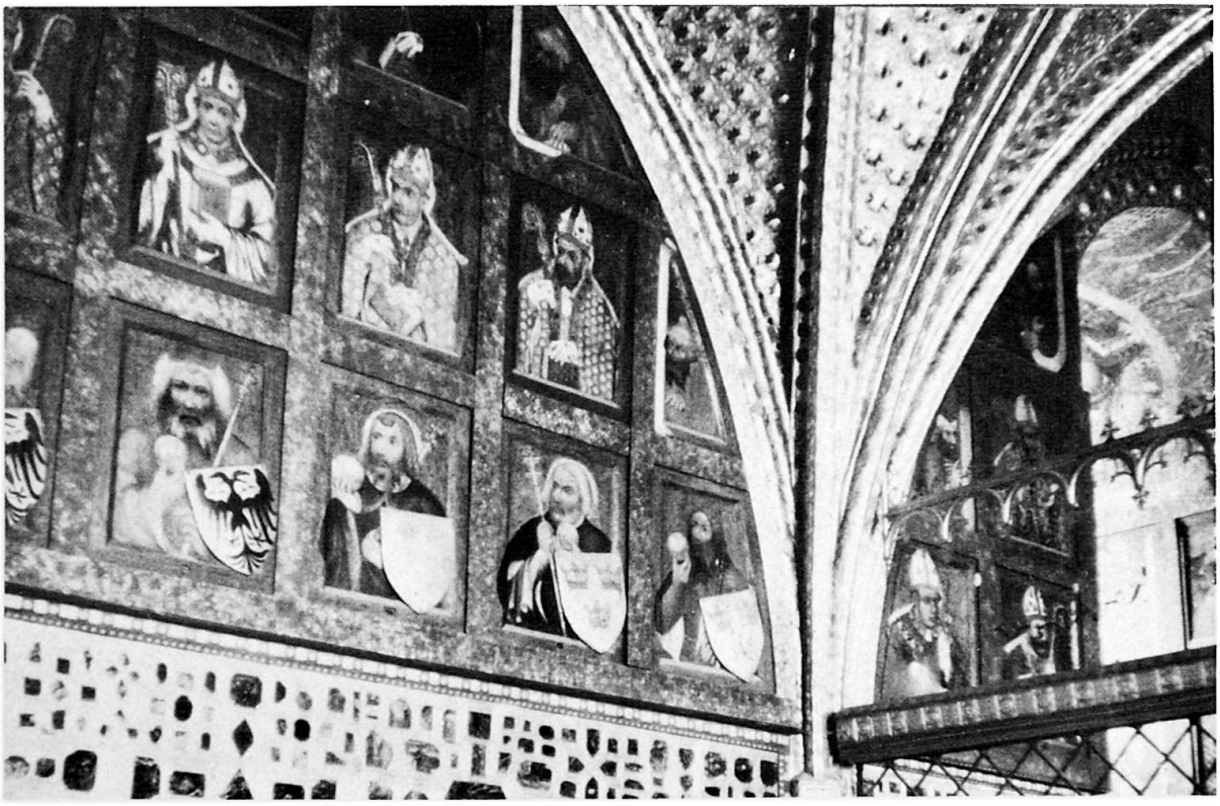

Fig. 2 Interior of the Chapel of the Holy Cross

The restoration of the Castle, carried out in the years 1888 - 1904 removed almost all traces of later renovations, made toward the end of the 16th century. The ancient and venerable Castle, the real memorial of Bohemian national history was deprived during the restoration of the traces of its later rich life. It was artifically clothed into the garment of the 19th century but its interiors with costly paintings have remained mostly intact and represent thus the largest treasure of Gothic religious paintings in the whole world.

Lately it was decided to protect these paintings against the danger of fire and collapse of the 18 meters high roof trusses erected during the last reconstruction

at the end of 19th century. Design of fire- and impact- resistant ceilings, prepared by the author together with Mr. J. Bělohlávek, M.Sc., C.E., will be briefly shown in this paper.

2. Purpose and Construction of the Fire- and Impact- Resistant Ceilings

The organisations of state care of historical monuments of Czechoslovac Socialist Republic decided that two fire- and impact- resistant ceilings have to be build on Karlštejn, protecting the Chapel of the Holy Cross and the Chapel of Virgin Mary from fire and collapse of roof trusses. Both these ceilings are designed in the loft level /Fig. 3/, having three main purposes:

- to protect the lower floors under loft from fire which could possibly be caused by a thunder-bolt or other reason
- to shield the ceilings in lower floors from destroying caused by collapse of the burning wooden roof trusses
- and to prevent the lower floors from drenching through with liquids used to extinguish the fire.

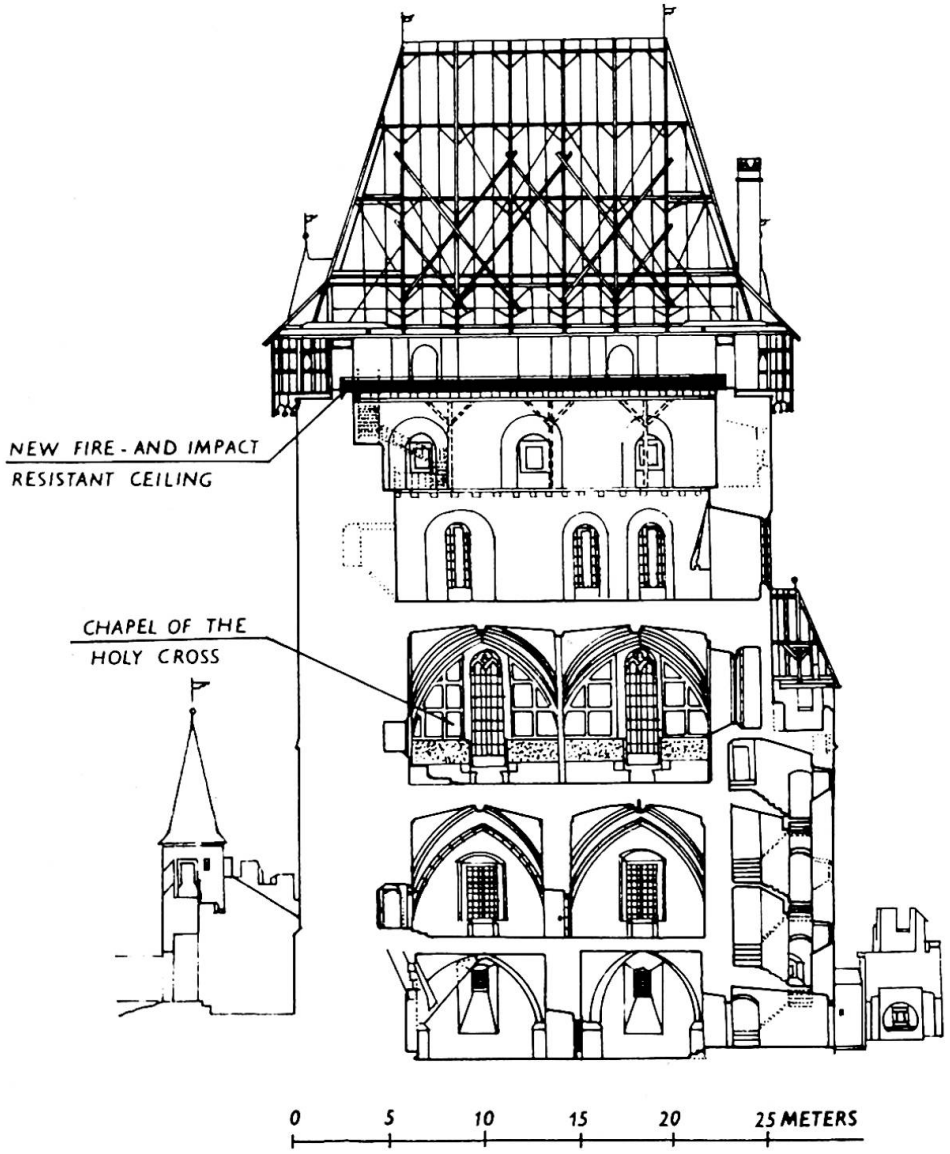

Fig. 3 Cross-section of donjon with position of the new ceiling

During the construction of these new ceilings no surcharge of old roof structures with low bearing capacity was permited. As can be seen the task was rather complicated in every respect, in particular taking into account the dynamic forces acting simultaneously with very high temperature, reaching more than 1200 °C. Solution of this problem was given by the author in form of a sandwich plate, where the upper layers have a heat insulation function damping at the same time the effects of impact. According to the thermal analysis in the first phase the roof crating with slate covering will be destroyed, falling all over the surface of the ceiling nearly at the same time. A protective net supported by cables anchored in

a special steel structure was designed to take this first dynamic load, falling down from the height of more than 15 meters and having a weight of nearly 40 Mp. This collapsed steel structure together with the ruins of the roof crating will form another damping layer retaining the impact effects of falling heavy parts of the roof truss.

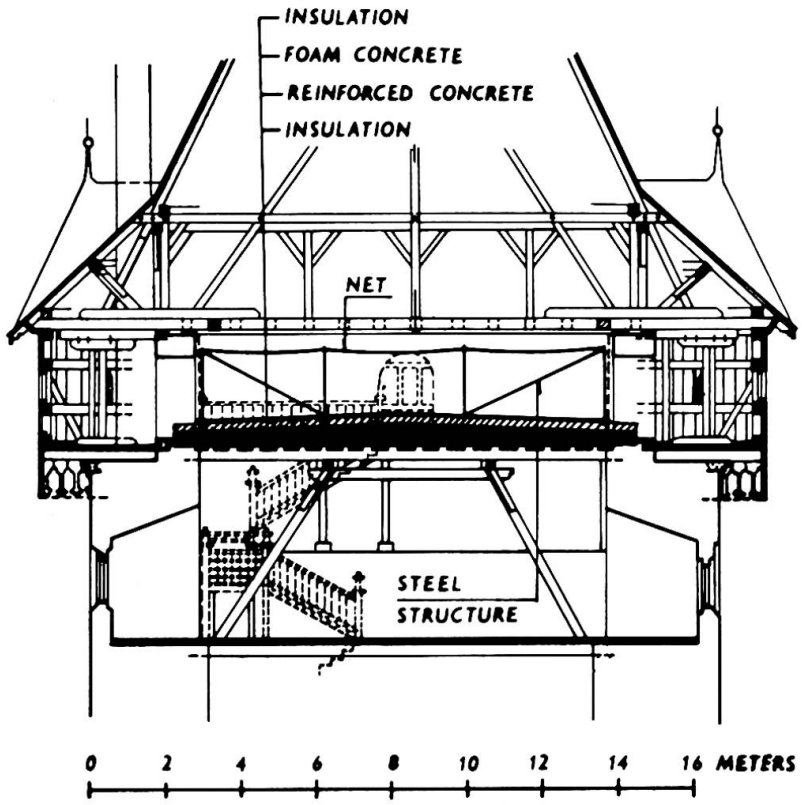

Fig. 4 Location and structural layers of the new designed ceiling

The main part of the new ceiling is formed by a reinforced concrete plate /Fig. 4/ of variable thickness placed on the outer bearing stone walls. As surcharge of old ceilings was not permitted it was designed to demolish the present brick paving with filling on the loft and to use the old battens as a formwork for the new ceiling. Both thermal and waterproofing insulation was proposed to be used on the lower surface, too, and in the lower room a steel centering was designed, taking the weight of the concrete placed in the first phase. The weight of the largest part of the new ceiling is carried by this first - step reinforced concrete structure which is concreted in an elevated position so that no load acts on the wooden old structure.

3. Thermal and Dynamic Analysis

The thermal analysis was made by Mr. V. Reichel, M.Sc.,C.E., as a basis for the next dynamic and static design and basic considerations. The full cubic contents of the roof truss and the crating is about 110 m^3 of wood and the floor space among the outer bearing walls is 221 m^2. The results of the thermal analysis are as follows:

- the collapse of the roof crating will take place at about 20 minutes from the time when the fire starts
- the full breakdown of roof truss will last about 90 minutes
- the weight of the falling parts of the roof truss will be about a fourth part of the original weight, which is due to burning
- the temperature history during the fire on the top of the fire-resistant ceiling as calculated is shown in Fig. 5
- the relationship between fire duration and temperature in the concrete slab when no upper heat insulation layers were assumed shows Fig. 6.

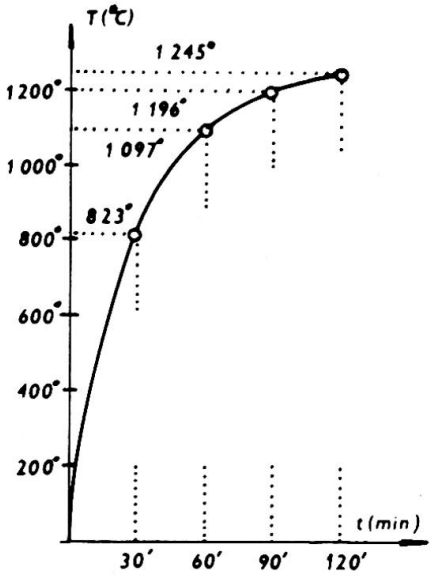

Fig. 5 Calculated temperature history during the fire

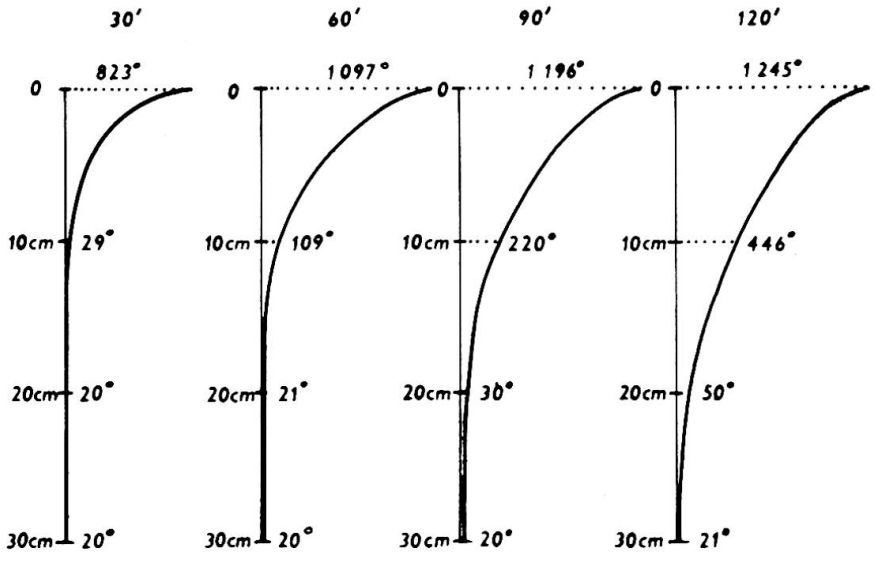

Fig. 6 Relationship between fire duration and temperature in the concrete without heat insulation layers

The dynamic analysis was made by Mr. Z. Podrásky, M.Sc.,C.E., in the way shown by Koloušek /5/ and on the basis of the results of thermal analysis. In the present study only a simplified dynamic design took place, using an energy method with reduced mass of the slab calculated by means of net method. The impact factor was calculated from the expression

$$\delta^{\sim} = 1 + \sqrt{1 + \frac{2h}{v_0} \cdot \frac{1}{1 + \frac{G_{red.}}{G_o}}} \qquad /1/$$

where are:

$\delta^{\sim}$ ..... impact factor
h ..... height of the free fall
v_0 ..... statical deflection of the slab due to the load G_o
G_{red} ..... reduced mass of the slab
G_o ..... the mass of the falling part of the roof truss

The damping was considered in this design approximately by reducing the height and mass of the falling parts of the roof truss which cannot be, however, determined precisely. The value of the impact factor calculated in this way was 6,2.

4. Conclusion

The design of the fire- and impact- resistant ceilings on the Karlštejn Castle proposed by the author - when accepted - shall be realized in the years 1977 - 78. As many problems dealt with are quite new - some of them formulated here for the first time - much effort must be exerted to come to a complex and satisfactory solution. It shall be not only a question of more precisely defined assumptions and detailed calculation but also a task of much work on construction

details, material transport and, last but not least, better understanding of the philosophy of construction of this grand medieval building.

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SUMMARY

In the paper a design of fire- and impact-resistant ceilings on Karlstejn Castle is briefly described. In this medieval complex the largest collection of Gothic religious paintings in the whole world is deposited and the new ceilings have to protect this treasure against the danger of fire and collapse of the high wooden roof trusses erected during the reconstruction at the end of the last century. This complex problem was solved by designing a sandwich plate and some results of thermal and dynamic analysis of this structure are presented.

RESUME

L'auteur décrit la conception des plafonds résistant aux chocs et à l'incendie du château de Karlstejn. Cette construction médiévale possède la plus grande collection au monde de peintures religieuses gothiques et il fallait protéger celle-ci contre le danger d'incendie et d'effondrement des poutres du plafond en bois datant du siècle dernier. Ce problème a été résolu à l'aide d'un panneau sandwich; quelques résultats de calculs thermiques et dynamiques sont donnés.

ZUSAMMENFASSUNG

Der Entwurf von stoss- und brandsicheren Decken für Schloss Karlstejn wird vorgestellt. Dieses mittelalterliche Gebäude besitzt die grösste Sammlung gotischer Devotionsbilder der Welt; es war deshalb nötig diese gegen die Brand- und Zusammenbruchgefahr von Deckenholzträgern des letzten Jahrhunderts zu schützen. Das Problem wurde mit Sandwichplatten gelöst; einige Ergebnisse der thermischen und dynamischen Berechnung werden dargestellt.

Considérations sur l'étude théorique du comportement à l'incendie des structures en béton armé

Theoretische Betrachtungen zum Brandverhalten von Stahlbetonbauten

Considerations on the Theoretical Study of Reinforced Concrete Structures under Fire

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1. INTRODUCTION.

Le calcul au feu des structures en béton armé est un problème très complexe. D'une part, l'étude de la propagation de la chaleur dans les éléments en béton est difficile, par suite de la grande massiveté des sections droites et de la faible conductivité thermique du béton. D'autre part, le comportement des structures en béton armé est complexe, par suite de la fissuration et du comportement non linéaire du béton, celui-ci étant de plus fonction de la température. C'est pourquoi nous estimons que l'étude théorique de ce problème doit se faire à partir de techniques de calcul évoluées, telles que celles décrites dans le Rapport Préliminaire [1],[6].

Dans cet article, on présente un code de calcul basé sur la méthode des éléments finis, qui est particulièrement bien adaptée à la résolution de ce type de problème. Cependant, nous avons adopté une discrétisation plus poussée, de manière à prendre en compte les différences de comportement du matériau.

2. CALCUL DE L'EVOLUTION DE LA TEMPERATURE DANS LES ELEMENTS.

Le problème de la répartition de la température dans le béton a une importance considérable. En effet, si la distribution de la température est mal évaluée, le calcul de la résistance au feu sera automatiquement erroné.

Dans le Rapport Préliminaire [4], nous avons indiqué les méthodes utilisées pour résoudre ce problème. Dans le cas des structures en béton, il n'est pas possible de procéder aux mêmes simplifications que dans le cas des structures en acier [2], et il est nécessaire de recourir aux techniques numériques décrites dans la référence précitée.

En ce qui concerne les propriétés thermiques du béton, il faut noter que la conductivité thermique λ décroît avec la température. Des valeurs de 1,5 ou 1,6 kgcal/m.h.°C sont couramment admises à 0°C, tandis qu'elles tombent à 0,8 vers 1000°C. Il faut cependant signaler que des expériences récentes effectuées à l'Université de Gand ont conduit à des valeurs de λ beaucoup plus élevées aux températures modérées (2,8 pour le béton de gravier). La dilatation thermique du béton α augmente avec la température jusqu'à 400°C environ, puis elle ne varie plus guère. La capacité thermique c ne varie guère avec la température, sauf aux environs de 100°C, par suite des phénomènes endothermiques apparaissant à cette température. Néanmoins, il est assez malaisé de tenir compte de cette particularité dans un modèle mathématique.

3. CALCUL DE LA RESISTANCE AU FEU DES POUTRES EN BETON ARME

Le code de calcul utilisé est basé sur la méthode des éléments finis (cfr. figure 1). Une discrétisation supplémentaire est obtenue en divisant l'élément de structure en petits éléments parallélipipédiques (figure 2). Le comportement peut varier d'un élément à l'autre, ce qui permet de tenir compte des différentes hétérogénéités. On obtient alors une discrétisation très poussée, et la technique de calcul est assez proche de celle utilisée par le Professeur BRESLER et le "Fire Research Group" de Berkeley [1]

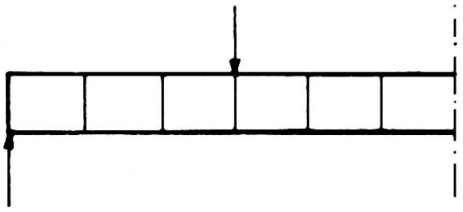

Figure 1

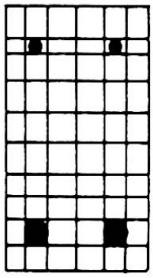

Figure 2

A partir des résultats fournis par l'étude thermique mentionnée au paragraphe précédent, il s'agit de calculer les déformations progressives de la poutre jusqu'au moment où l'on atteint la ruine par effondrement du système.

Deux phénomènes sont à la base de cette augmentation progressive des déformations :

1) L'augmentation de la température entraîne des effets thermiques qui dépendent fortement du mode d'appui de la poutre. Il faut noter que, dans le cas des structures en béton et contrairement à ce qui se produit pour les structures en acier, il existe toujours un gradient thermique important sur la section droite, par suite de la mauvaise conductivité thermique du béton.

   S'il s'agit d'une poutre sur deux appuis simples, il apparaît des dilatations thermiques importantes qui consistent en :

   - un allongement de la pièce ;
   - des déformations de flexion.

   Ces déformations thermiques se font sans apparition de contraintes dans le matériau si la distribution de la température sur la section droite est linéaire ; dans le cas contraire, il y a toujours apparition de contraintes thermiques, mais celles-ci restent généralement assez faibles.

   Si, au contraire, il s'agit d'une poutre biencastrée, des contraintes thermiques importantes sont induites dans la structure, par suite de l'apparition de moments et d'efforts normaux de bridage aux appuis. Les déformations thermiques, par contre, restent faibles.

   Dans le cas d'un élément isostatique, les calculs sont basés sur les formules de la thermoélasticité. Considérons une poutre de section quelconque, à comportement élastique, soumise à une distribution arbitraire de température sur la section droite. Les contraintes thermiques en un point d'ordonnée z sont données par la formule :

$$\sigma = -\alpha E \Theta + \frac{P}{A} + \frac{Mz}{I} \qquad (1)$$

avec A = aire transversale

I = moment d'inertie

α = coefficient de dilatation thermique

P et M ont pour expression :

$$P = \int_A \alpha E \Theta \, dA \qquad M = \int_A \alpha E \Theta z \, dA \qquad (2)$$

Figure 3

En ce qui concerne les déplacements dus aux effets thermiques, ils sont identiques à ceux de la même poutre soumise à une charge axiale P et à des moments d'extrémité M.

Dans le cas de la discrétisation envisagée, les formules précédentes doivent être modifiées,pour tenir compte du fait que les propriétés varient d'un point à l'autre en fonction de la température. Les expressions donnant P et M deviennent :

$$P = \sum_{i=1}^{p} \alpha_i E_i \Theta_i A_i \qquad M = \sum_{i=1}^{p} \alpha_i E_i \Theta_i z_i A_i \qquad (3)$$

où p désigne le nombre d'éléments parallélipipédiques.

Il faut noter ici que P et M ne sont pas rigoureusement constants le long de la poutre. De plus, comme le comportement du matériau n'est pas élastique, le calcul doit être fait pas à pas, en l'envisageant comme une succession de calculs élastiques.

2) L'augmentation de la température sur la section droite amène aussi une diminution des propriétés mécaniques du béton et de l'acier (résistance et module d'élasticité). Il en résulte que, sous charge constante, les déformations du système augmentent et la ruine se produit lorsqu'on dépasse la capacité portante de la poutre.

La prise en compte de ce phénomène ne présente pas trop de difficulté grâce à la discrétisation adoptée. En effet, la rigidité de chaque élément $[K]$ est la somme des rigidités des p parallélipipèdes $[K_i]$. Chacune de ces rigidités dépend du module d'élasticité du matériau qui est fonction de la température $E_i = E(\Theta_i)$: il diminue lorsque la température augmente. Il en résulte que les rigidités $[K_i]$ diminuent, donc aussi la rigidité $[K]$ de chaque élément et la rigidité globale de la structure.

Le calcul des rigidités est effectué par un procédé d'intégration numérique.

4. EXEMPLE D'APPLICATION

Les propositions précédentes sont illustrées par un exemple de calcul. Il s'agit d'une poutre sur deux appuis simples qui a été testée au Laboratoire Herpol de l'Université de Gand. Les dimensions de la pièce et le mode de chargement sont indiqués à la figure 4, tandis que les dimensions transversales et la position des armatures sont données à la figure 5.

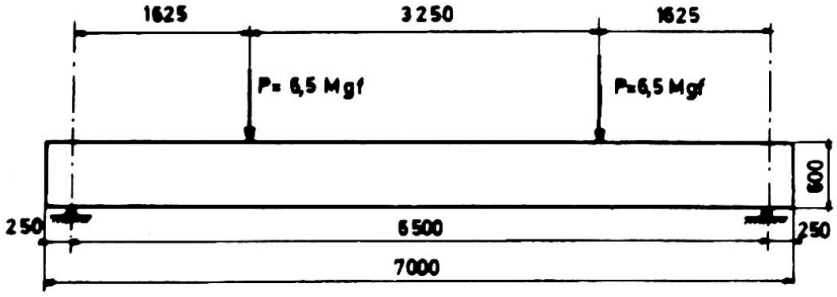

Figure 4

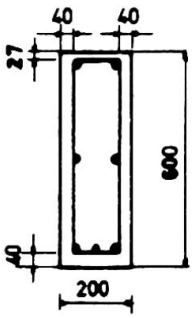

Figure 5

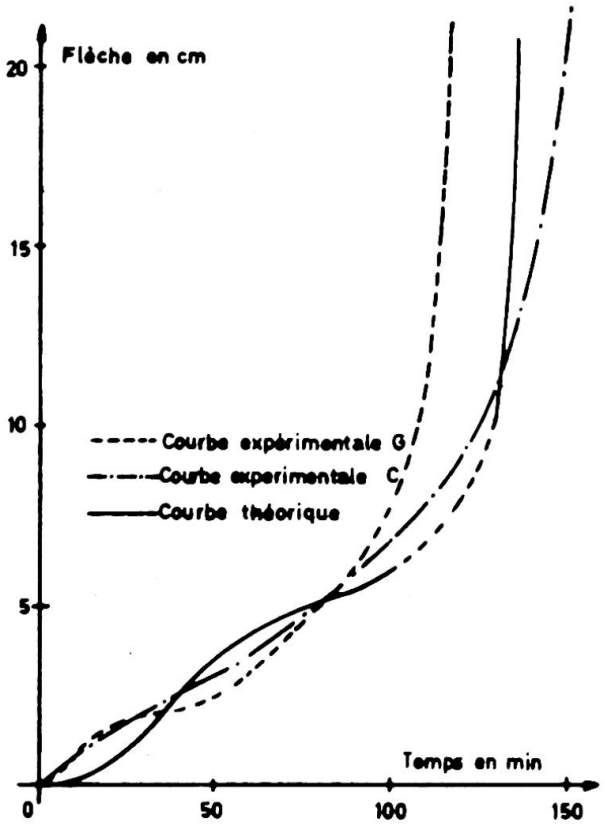

Figure 6

Une analyse par éléments finis a été réalisée pour la détermination de la distribution de la température (cf. [4]). On a noté une bonne concordance avec les résultats expérimentaux. La figure 6 donne l'évolution de la flèche en fonction du temps lorsque la température dans le four s'élève. Il y a deux courbes expérimentales, car deux types de béton ont été utilisés pour confectionner les poutres. On peut noter que les résultats obtenus avec notre modèle sont très satisfaisants.

5. CONCLUSIONS.

Les co,sidérations développées dans cet article montrent l'intérêt des méthodes numériques pour la prévision par le calcul du comportement à l'incendie des structures en béton armé. Les résultats obtenus pour les poutres isostatiques sont très encourageants.

En ce qui concerne les poutres hyperstatiques, des recherches sont actuellement en cours, car l'évolution des moments de bridage aux appuis semble dépendre fortement du fluage à chaud du béton.

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RESUME

On présente un modèle mathématique pour l'évaluation de la résistance au feu des éléments en béton armé. Ce modèle est basé sur la méthode des éléments finis, mais une discrétisation plus poussée est utilisée pour prendre en compte les différences de comportement des matériaux. Les résultats obtenus pour les poutres isostatiques sont très encourageants.

ZUSAMMENFASSUNG

Man stellt ein mathematisches Modell zur Untersuchung des Feuerwiderstandes von Stahlbetonelementen vor. Dieses Modell stützt sich auf die Methode der finiten Elemente; man benützt aber eine feinere Einteilung, um die Unterschiede im Materialverhalten berücksichtigen zu können. Für einfache Balken sind die erreichten Ergebnisse vielversprechend.

SUMMARY

A mathematical model is presented for the evaluation of the fire resistance of reinforced concrete elements. This model is based on the finite element method, but a more refined discretization is used to take into account the various material behaviours. The results obtained for isostatic beams are very promising.

Rôle de l'eau libre dans le béton soumis au feu

Wirkung des Freiwassers im brandbeanspruchten Beton

Action of Free Water in Concrete under Fire

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L'eau joue un rôle considérable dans le comportement des ouvrages en béton face au feu : on lui attribue aussi bien les éclatements qui peuvent conduire à la ruine que d'importantes absorptions de chaleur dues à la vaporisation qui permettent de prolonger de manière considérable la tenue au feu.

Tout dépend de la manière dont est répartie l'eau libre et de la manière dont elle pourra se vaporiser.

1. ETUDES EN COURS.

Peu d'études systématiques ayant été publiées à ce jour, nous avons entrepris trois séries de recherches :

1.1. L'une sur des dalles homogènes de 7 et 12 cm d'épaisseur soumises dans leur plan à une contrainte axiale de 80 daN, variant par :

- la nature des granulats (calcaire, silico-calcaire, siliceux),
- le rapport eau/ciment (0,40 - 0,65),
- la répartition et la nature des armatures (acier doux Ø 20 - acier mi-dur Ø 6 et 20).

1.2. L'autre sur des dalles composées d'une prédalle de 5 cm associée à 6 à 9 cm de béton coulé en place.

1.3. La troisième sur des teneurs en eau réelles des ouvrages en service.

2. AVANCEMENT DES TRAVAUX.

Les difficultés de réalisation de cette dernière recherche font que nous en sommes encore dans la phase préparatoire, par contre les deux premières nous ont déjà apporté des résultats intéressants, encore partiels (achèvement prévu dans un an), dont voici les grandes lignes :

2.1. Essais d'éclatement sous contrainte axiale.

Essai selon la courbe ISO effectué à partir de juin 1976 sur des dalles de 1,5 x 0,8 m âgées de 90 jours (figure 1).

Le hasard a voulu que les mois de juin et juillet 1976 aient été en France particulièrement secs aussi, lors des essais, l'eau libre était évaporée dans les proportions ci-dessous :

	PERTES EN POIDS en % du poids total à 90 jours			
Epaisseur	7 cm		12 cm	
Rapport initial eau/ciment	0,40	0,65	0,40	0,65
à 90 jours	1,44	1,65	1,07	1,26
pendant l'essai	3,00	2,88	2,33	2,27

pour les dalles constituées avec des granulats silico-calcaires du Bassin Parisien. Parmi celles-ci, une seule dalle a éclaté et ce, dans sa partie armée (les dalles sont armées sur la moitié de leur surface); ses caractéristiques étaient :

- épaisseur : 12 cm - e/c = 0,65 - acier Tor 6 mm (maille 10 x 15 cm) -

Nous avons noté :

2,11. A propos des déformations longitudinales (figure 2), qu'à durée égale :

- les dalles non armées se raccourcissent rapidement en prenant de la courbure,
- de même, les dalles se raccourcissent dès qu'il y a éclatement si elles sont armées,
- les dalles gâchées les plus sèches paraissent les plus rigides, et ce d'autant plus que l'armature est grosse et que la dalle est mince,
- le palier de dilatation est obtenu pour une température d'environ 275°C pour l'acier doux et 350°C pour l'acier écroui à froid.

2,12. A propos de la température des aciers (figure 3), qu'à durée égale :

- il y a un gain compris entre 40°C et 70°C quand l'épaisseur passe de 7 à 12 cm (il n'y a pratiquement pas de palier à 100°C pour 7 cm d'épaisseur), mais qui peut être plus fort du fait que si les aciers sont de plus gros diamètre, l'enrobage est plus important.
- il existe un gradient d'environ 70°C entre la génératrice inférieure et la génératrice supérieure des aciers de 20 mm (acier lisse ou non), et les températures sont du même ordre sur les aciers lisses et sur les aciers à adhérence améliorée,
- le rapport e/c joue à peine pour les dalles de 7 cm, mais permet de gagner environ 100°C pour les dalles de 12 cm.

2,2. Essais de prédalles (Figure 4).

Les dalles ont 4 x 1,8 m et sont chargées à 250 kg/m2. Les prédalles ont toutes 5 cm d'épaisseur, le béton coulé en place 6 cm en général.

Dès à présent, il convient d'insister sur l'importance que revêt la prédalle.

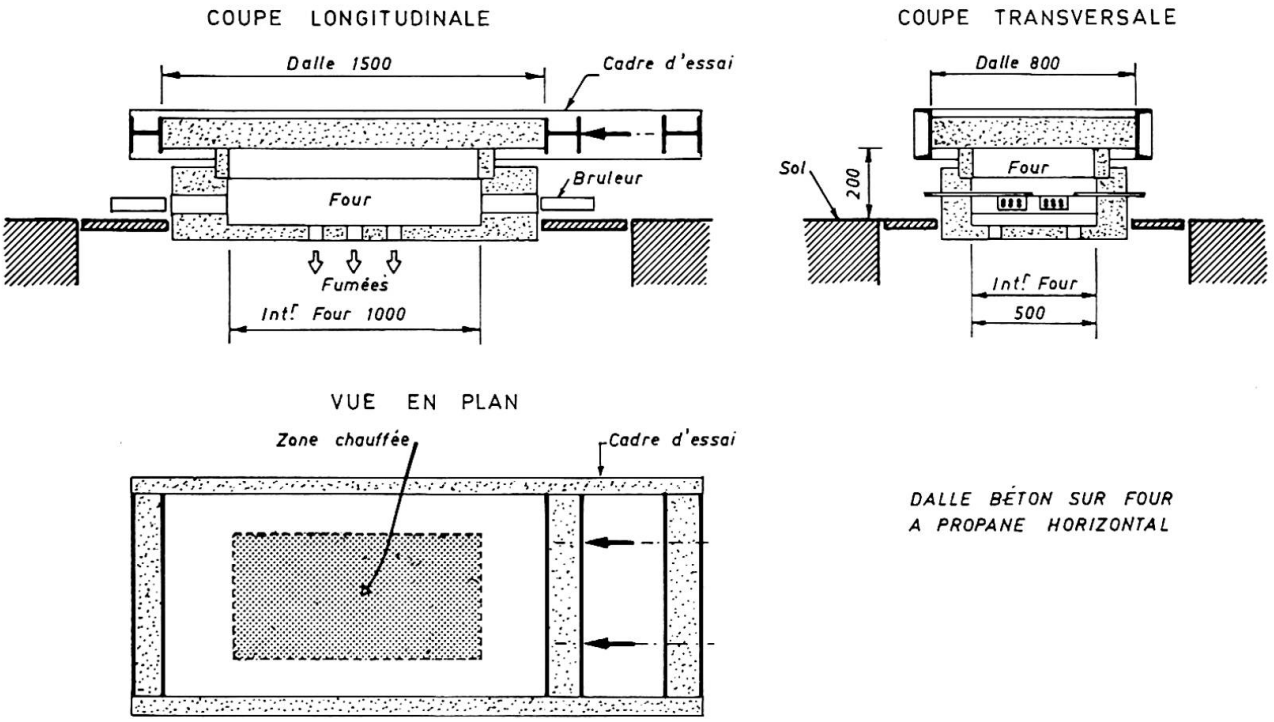
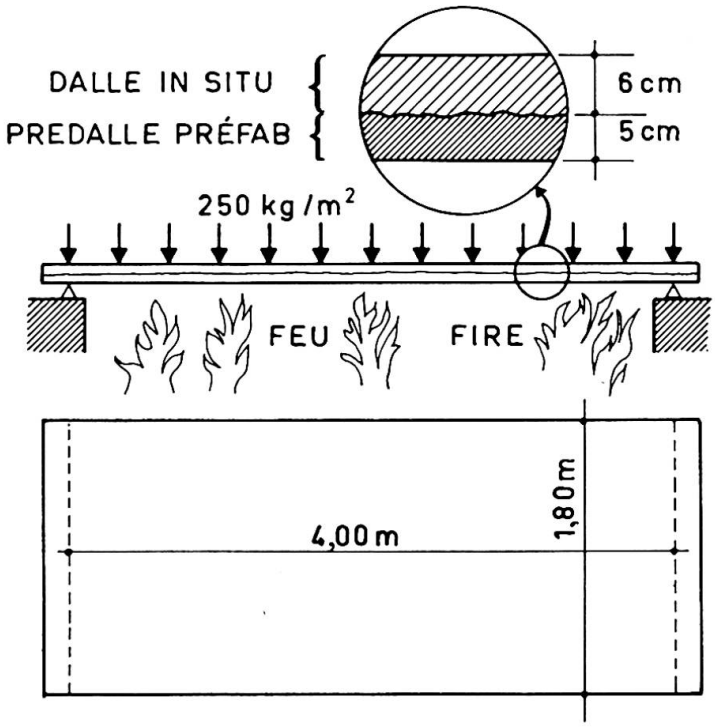
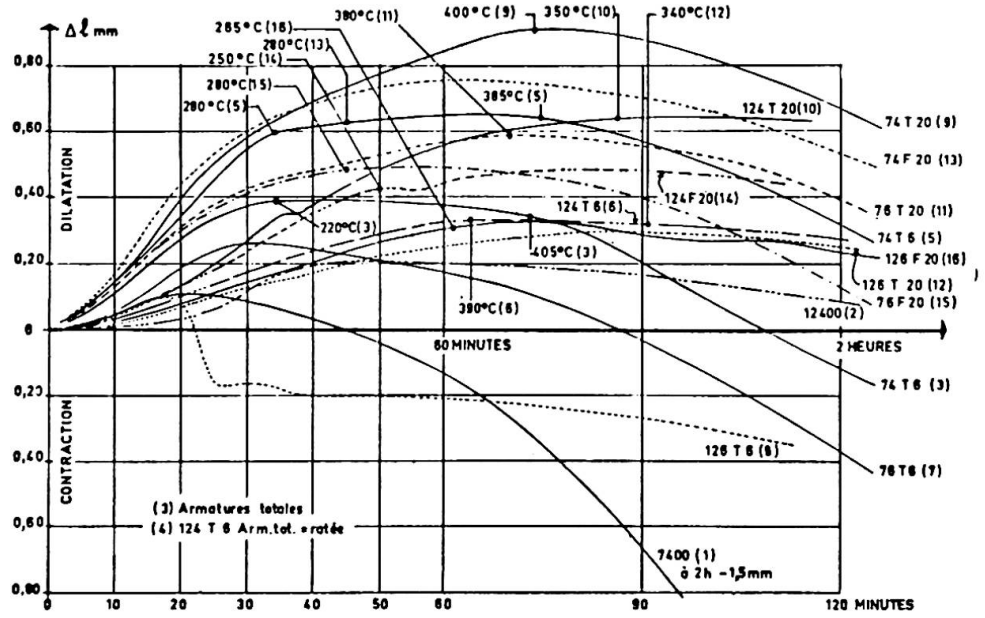


Fig. 1.– Schéma des dalles comprimées axialement et du dispositif d'essais.





28.7.76 ESSAIS ÉCLATEMENTS *CHAMPS SUR MARNE*.

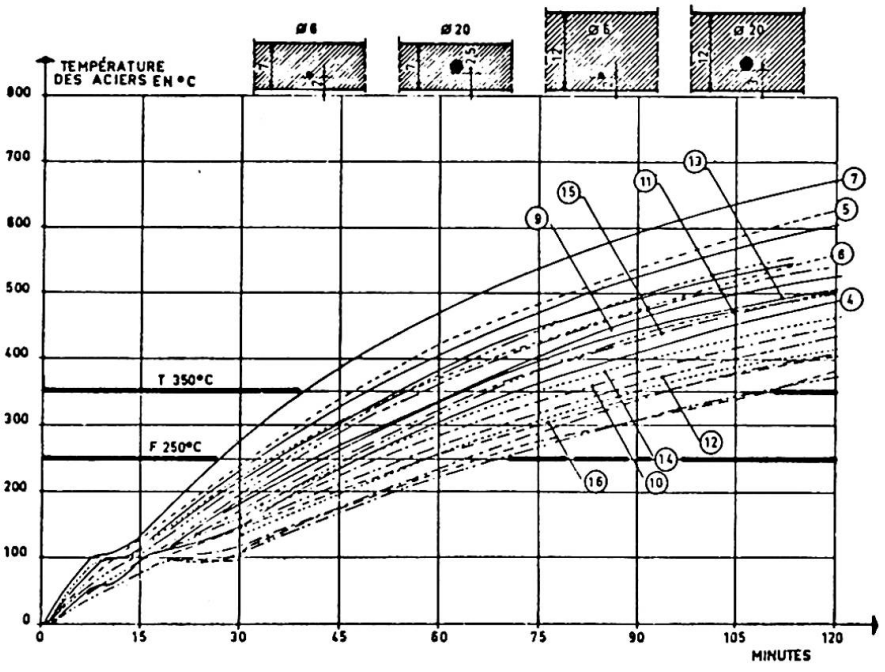


Fig. 2 et 3.- Essais d'éclatements de dalles comprimées axialement. Dilatation axiale et température des aciers :

- premier nombre (7 - 12) : épaisseur en cm
- deuxième nombre (4 : e/c = 0,40 ; (6 : e/c = 0,65
- troisième nombre (T 6 : acier écroui Ø 6 mm ; (T 20 : acier écroui Ø 20 mm ; (F 20 : acier doux Ø 20 mm

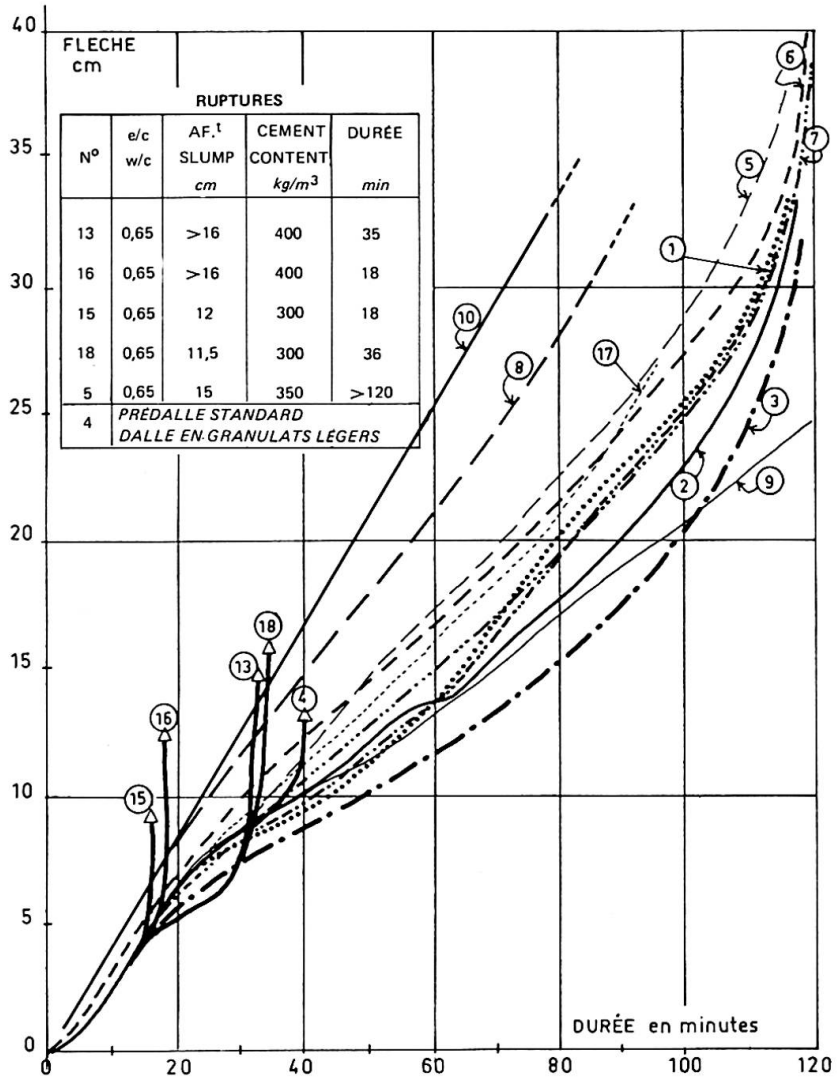


Fig. 5. Essais de prédalles (bi-couches) : Mesure des flèches.

Repérage des échantillons :

1. Elément standard.

(prédalle : 5 cm - silico-calcaire - e/c = 0,45 - moule vibrant -
(350 kg/m3 de CPA 400 étuvé à 60°C - armature Ø 12 tous les 19 cm.
(dalle : 6 cm - silico-calcaire - e/c = 0,50 - aiguille vibrante -
(350 kg/m3 de CPAL 325 non chauffé.

Repère	Autres échantillons	Paramètre
2.	prédalle = 0 cm, dalle = 11 cm	épaisseur
3.	prédalle = 5 cm, dalle = 9 cm	
5.	prédalle non vibrée (e/c = 0,65)	Mode de serrage
6.	prédalle aiguille vibrante (e/c = 0,45)	
7.	prédalle moule vibrant (e/c = 0,45)	
4.	prédalle = 5 cm, dalle = 11 cm de granulats légers	Type de granulats
8.	prédalle granulats silico-calcaires	
9.	prédalle granulats calcaires	
10.	prédalle granulats siliceux	
11.	prédalle - armature ϕ 16 tous les 33 cm	Armature
12.	prédalle - armature Ø 8 tous les 9 cm	
13.	Dalle e/c = 0,50 Prédalle 400 kg/m3 de CPA 400 e/c = 0,65	Ciment et e/c
14.	" e/c = 0,50 " 300 kg/m3 de CPA 400 e/c = 0,45	
15.	" e/c = 0,50 " 300 kg/m3 de CPA 400 e/c = 0,65	
16.	Dalle e/c = 0,65 Prédalle 400 kg/m3 de CPA 400 e/c = 0,65	
17.	" e/c = 0,65 " 300 kg/m3 de CPA 400 e/c = 0,45	
18.	" e/c = 0,65 " 300 kg/m3 de CPA 400 e/c = 0,65	

Ainsi nous avons constaté (figure 5) :

- à un dosage en ciment portland de 350 kg/m3, une prédalle non vibrée à la fabrication entraîne des flèches importantes à partir de la première heure et la ruine après 2 heures lors du refroidissement ;
- le type de granulat utilisé pour la prédalle joue un rôle important : ainsi pour atteindre une flèche de 1/30e, le temps est passé de 30 minutes à une heure lorsque l'on remplace la silice par du calcaire ;
- quel que soit le dosage (300 ou 400 kg/m3 de ciment portland), un rapport eau/ciment de 0,65 entraîne une rupture brutale très rapide au cours de la première demi-heure de l'essai, alors que la sous-face est entre 260 et 300°C, les aciers entre 115 et 125°C, l'interface entre 60 et 90°C et la surface entre 18 et 20°C. La rupture s'explique par la désolidarisation complète à l'interface ;
- une surface d'accrochage insuffisamment rugueuse constitue un chemin préférentiel pour l'eau, et si certaines zones lisses sont entourées de parties rugueuses, cela peut provoquer un décollement brutal des deux couches.

D'ici un an, nous pourrons grâce en partie à ces résultats compléter le règlement actuellement en vigueur en France concernant le calcul des ouvrages au feu (D.T.U. "Méthode de prévision par le calcul du comportement au feu des structures en béton").

RESUME

L'intervention traite du rôle de l'eau libre se trouvant dans le béton au moment de l'incendie dans les deux cas suivants:

- Dalles homogènes de 7 et 12 cm soumises à une contrainte de compression unidirectionnelle dans leur plan de 80 daN,
- Dalles isostatiques constituées de 2 couches de béton d'âges différents chargées uniformément.

Les recherches en cours montrent l'incidence sur la température des armatures et leur comportement selon qu'il s'agit d'aciers doux ou écrouis, l'importance d'un dosage suffisant en béton (350 kg/m3), de la qualité de la mise en oeuvre et de la qualité de la liaison entre les deux couches de béton.

ZUSAMMENFASSUNG

Der Artikel behandelt die Wirkung des Freiwassers im Beton unter Brandbeanspruchung in den zwei folgenden Fällen:

- Homogene Platten, 7 und 12 cm stark, die einer eindimensionalen Druckspannung von 80 daN in ihrer Ebene unterworfen sind,
- Statisch bestimmte Platten unter Gleichlast, die aus einem 5 cm starken vorgefertigten Element und einem Aufbeton bestehen.

Die laufenden Untersuchungen zeigen den Einfluss der Bewehrungen auf die Temperatur und ihr Verhalten je nachdem, ob es sich um naturharten oder um kaltverformten Stahl handelt, die Bedeutung einer genügenden Zementdosierung (350 kg/m3), die Verarbeitungsgüte und die Güte der Haftung zwischen den beiden Betonlagen.

SUMMARY

The paper concerns the action of free water in concrete during a fire into the following cases:

- Uniform slabs of 7 and 12 cm thickness submitted to an axial load in their own plan of 800 N,
- Isostatic slabs formed by a layer of concrete poured in situ on a precast 5 cm thin slab with uniform loading.

The investigation in progress shows the action of the reinforcing bars on the temperature and on their behaviour in relation with the type of steel bars, the importance of the cement contents (more than 350 kg/m3), of the water/cement ratio, and of the good quality of the bonding between the two concrete layers.

Spalling of Concrete in Actual Fire

Eclatement du béton dans un incendie réel

Ausplatzen von Beton bei einem echten Brand

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Spalling experiment conducted with fifty-two 1m x 1m test panels of reinforced concrete has shown a prominent effect of free water in concrete on spalling phenomenon of both lightweight and ordinary aggregate concrete, as we have reported in the preliminary report. The result shows that spalling is liable to occur when the amount of free water in concrete is increased above the range of 120 to 130kg per cubic meter of concrete. 1)

Fig. 1 and fig. 2 show the examples of test panels exposed to fire. The test panel in fig. 1 made of ordinary aggregate concrete and having 140kg/m^3 of free water shows a remarkable spalling, while the panel in fig. 2 made of lightweight aggregate concrete shows a slight spalling on the exposed surface, the amount of free water being 120kg/m^3.

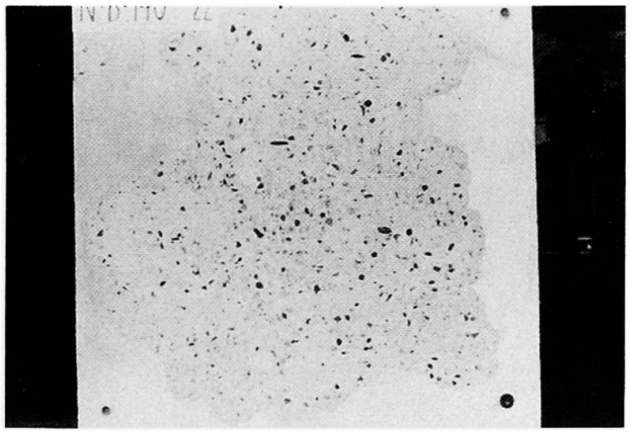

Fig.1
Ordinary aggregate concrete test panel having 140 kg/m^3 of free water

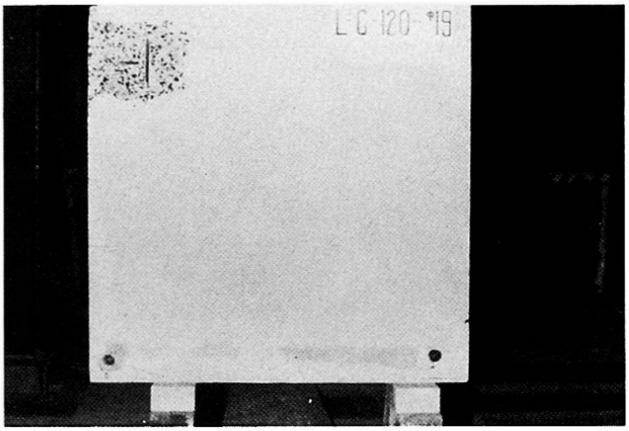

Fig.2
Lightweight aggregate concrete test panel having 120 kg/m^3 of free water

An investigation is now under consideration to make clear what amount of free water exists in realized concrete structures. A preliminary study for this investigation on the loss of free water from ø15 x 30cm cylinder made of lightweight aggregate concrete in the course of atmospheric drying in the room shows that lightweight aggregate concrete has about 100 to 120kg/m^3 of free water at the age of 3 months, and the amount of free water of air dried concrete seems to be in equilibrium within this level (fig. 3).

We can consider that the amount of free water in the lightweight aggregate concrete in realized structures also decreases to this level after the age of 3 months or more, according to the size and exposure conditions of the concrete elements. For ordinary concrete, the initial and the equilibrium amount of free water is less than those for lightweight aggregate concrete.

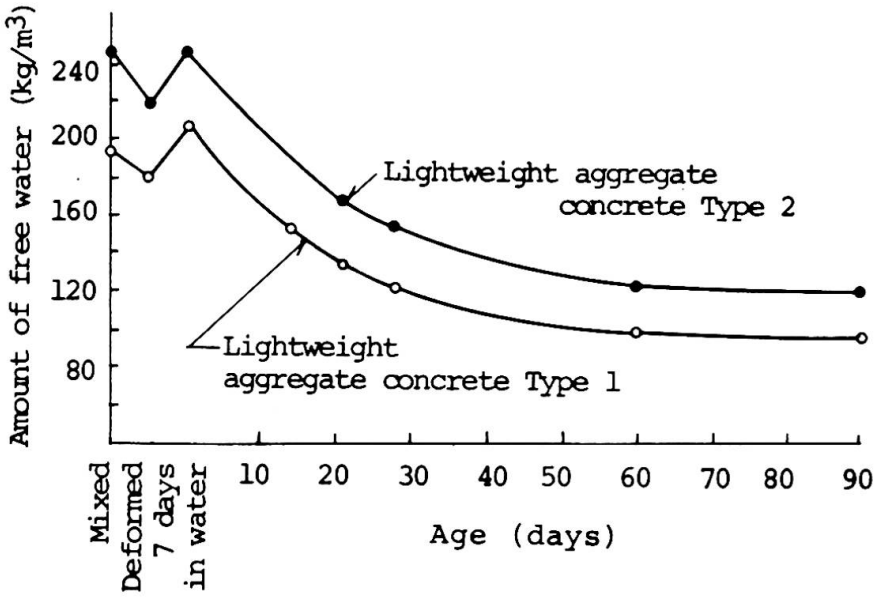

Fig. 3 Loss of free water from lightweight aggregate concrete (ø15 x 30 cylinder)

The following are some examples of spalling phenomenon in actual fire which seem to represent different levels of the amount of free water and different types of aggregate used. The examples are limited but we could recognize an effect of the amount of free water in concrete on spalling in actual fire, as in the experiment.

Example 1: Explosive Spalling Occurred in Fire of a Reinforced Concrete Structure Building under Construction

This example, reported by K. Kizawa and S. Ohgishi in 1967, shows a remarkable spalling in concrete during hardening. A five-story reinforced concrete shop building having 2,200m^2 total floor area was under construction when fire spread from the 3rd to the 5th floor, 5 days after concreting in the 4th story and floor slab of the 5th story. It was 17th day after concreting in the 3rd story.

Spalling was so much intensitive as to be explosive at the 5th story floor slab and at the bearing wall around the elevator shaft in the 3rd and 4th story. The concrete was of ordinary aggregate

and the amount of free water should have been very high because of early ages of concrete (fig. 4).

Fig.4
Spalling at the wall of the elevater shaft

Example 2: Spalling in Fire of a Reinforced Concrete and Steel Structure Building Nearing Opening

The building was a six-story steel structure with a basement of reinforced concrete, having 60,000m^2 total floor area. The floor slab of each story was of reinforced lightweight aggregate concrete except that ordinary aggregate concrete was used in the basement.

Fire occurred at the basement, just before the opening of the building, spread from the 2nd to the 4th floor, the 1st floor remaining undestroyed, and continued for 15 hours smoldering. It was about 6 months after concreting.

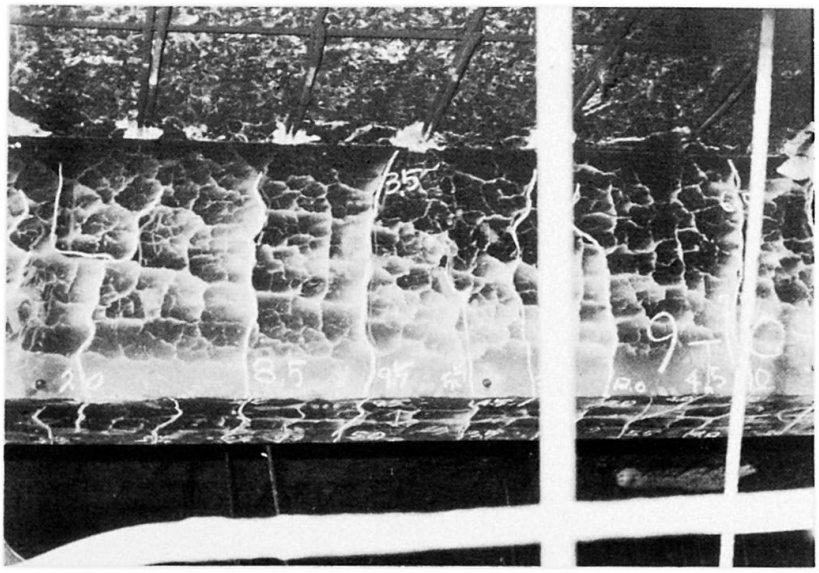

Fig.5
Spalling at the under side of floor slab

Spalling of concrete was observed at the under side of each floor slab and at the reinforced concrete beam and girdir of the basement. It was so much remarkable in severely attacked slabs that the lower reinforcing bars of the slabs were entirely bared out (fig. 5).

It was considered that the cause of this remarkable spalling could be a fairly high content of free water in the concrete, especially in that of the basement, and the long duration of fire due to the abandance of combustibles.

Example 3: Spalling in Fire after Gas Explosion of a Precast Concrete Structure Apartment House

The building was of a composite structure of H section steel frame and precast concrete large panels. The large panels were made of lightweight aggregate concrete and manufactured by steam curing. Fire occurred after gas explosion in an apartment on the 6th floor and spread to adjoining 5 apartments.

Though the damage by the gas explosion was so heavy as to destroy the walls and slabs (fig. 6), spalling was observed only at the lower corner of the beams and around the ventiration opening penetrating them (fig. 7). The cause of the slight spalling might be a fairly low content of free water in the concrete due to steam curing.

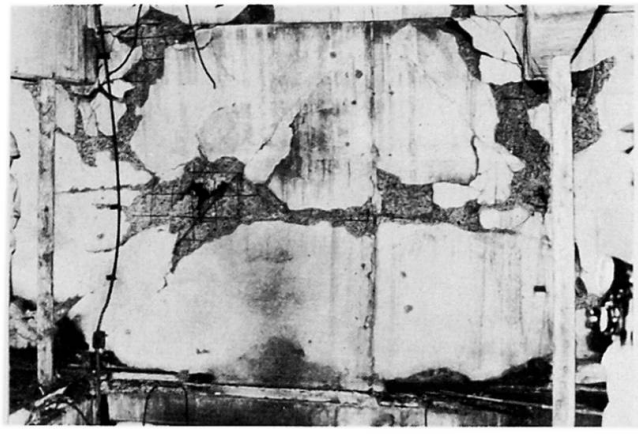

Fig.6
A wall destroyed by gas explosion

Fig.7
Spalling at the beam

Example 4: Spalling in Fire of a Reinforced Lightweight Aggregate Concrete Apartment House

The apartment house is of a composite structure of lightweight aggregate concrete cast-in-situ, having 11 stories. Fire occurred in March 1976, i.e. 6 years after completion, in an apartment on the 8th floor and this apartment was fully burnt out.

Spalling could scarcely be observed except at the under side of balcony slab of the upper story. The spalled part was about 80cm x 50cm, depth about 2cm, and its appearance was as same as that of test panels spalled in our experiment. This part has been attacked directly by the flame going out from the broken window, and the amount of free water in the slab should have been higher than in interior elements (fig. 8).

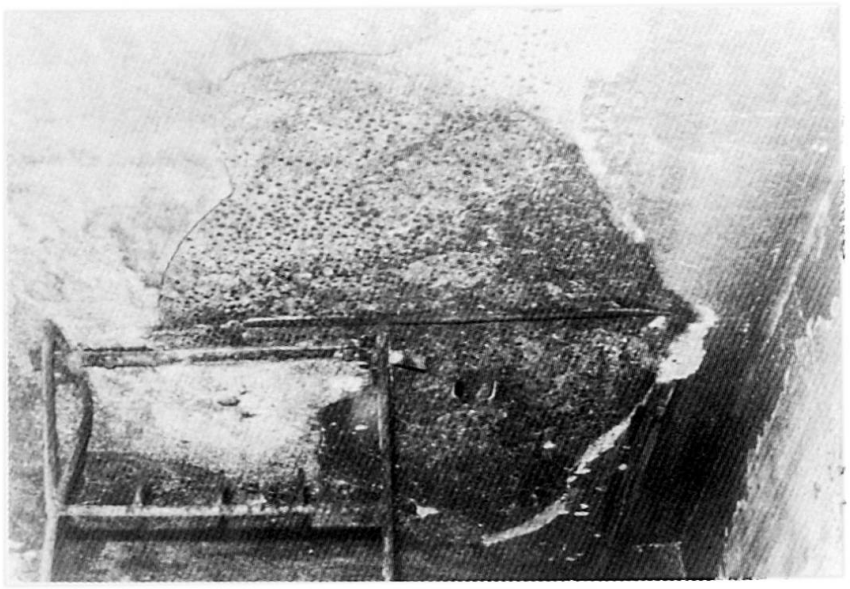

Fig.8
Spalling at the under side of balcony slab

SUMMARY

Four examples of spalling of concrete in actual fire representing different levels of the amount of free water and different types of aggregates used are shown. An effect of the amount of free water in concrete on spalling could be significant in actual fire as in the experiment, the amount of free water in realized concrete structures being to be investigated.

RESUME

On présente 4 exemples d'éclatement du béton dans un incendie réel, avec des bétons correspondant à différentes quantités d'eau libre et à des agrégats différents. L'effet d'une certaine quantité d'eau libre pourrait être déterminant pour l'éclatement du béton, dans des incendies réels comme en laboratoire, mais il reste à déterminer la quantité d'eau libre se trouvant en réalité dans le béton des structures existantes.

ZUSAMMENFASSUNG

Vier Beispiele des Ausplatzens von Beton bei einem echten Brand werden gezeigt; die Betons werden mit verschiedenen Mengen Freiwasser und mit verschiedenen Aggregaten vorbereitet. Die Wirkung einer gewissen Menge Freiwasser auf das Ausplatzen des Betons könnte sowohl bei wirklichen Bränden wie auch im Labor entscheidend sein, es wäre aber noch notwendig, die Menge Freiwasser im Beton von bestehenden Bauwerken festzustellen.

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IV

Constructions spéciales
(acier, béton, mixtes; études comparatives)

Spezielle Bauwerke
(Stahl, Beton, Verbund; vergleichende Studien)

Special Structures
(Steel, Concrete, Composite; comparative Studies)

IVa

Constructions en mer

Bauwerke im Meer

Offshore Structures

IVb

Structures des fondations pour les maisons hautes

Fundationen für Hochhäuser

Foundation Structures for Tall Buildings

IVc

Structures des fondations pour les ponts de grande portée

Fundationen für weitgespannte Brücken

Foundation Structures for long span Bridges

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Comments by the Author of the Introductory Report

Remarques de l'auteur du rapport introductif

Bemerkungen des Verfassers des Einführungsberichtes

J.G. BOUWKAMP
President
Offshore Development Engineering, Inc.
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Offshore Structures

Mr. Chairman, Ladies and Gentlemen,

The contributions to the theme, Offshore Structures, as published in the Preliminary Report, covered not only general design aspects of both steel and concrete structures, but also focused on at least one of the more significant design details, namely tubular joints.

The contributions by Roret and Ciolina on steel structures, and Gerbault and Xercavins on concrete structures, give an up-to-date general review of several structural systems which have been developed for the exploration and production of hydrocarbon deposits in offshore regions. Unfortunately, space limitations prevented discussion in depth of detailed technical problems associated with the design, construction and installation of these structural systems. Most problems are related to the dynamic behavior of the structures under the effects of waves, currents, wind, ice and earthquakes, and require full consideration of the soil-foundation-structure-fluid interaction. For instance, soil conditions are a major factor in evaluating both the short- and long-term dynamic response of gravity platforms under cyclic, wave-induced loads. Equally significant is the dynamic behavior of steel tubular trusses and their connections under such load conditions. In addition to the response to extreme waves and winds, earthquakes are another important factor. One of the most significant elements in the structural design of steel tubular trussed towers is the tubular joint. It was, therefore, particularly fortunate that the contributions of Kurobane, et al, and Okumura, et al, were addressed to the performance of tubular joints. In light of my own experience in this subject, I would like to focus on this particular aspect in greater depth. However, first I would like to note the contributions by Shimada and Yamamoto, and Coulard. Although the authors focused on harbor and coastal structures, rather than offshore structures, they do draw attention to the fact that offshore developments are not complete without the necessary onshore facilities for equipment maintenance and transfer of oil and gas to shore.

SB

The history of tubular joint fatigue research goes back almost twenty years. At that time the primary focus was directed towards developing joint design criteria which would assure the overall structural integrity of a tower structure under extreme wave and wind conditions.(1,2,3,4) These criteria reflected the environmental conditions in the relatively shallow waters of the Gulf of Mexico where extreme waves were the critical design condition. The resulting design criteria based on the "punching shear" strength concept resulted in joints which were capable of withstanding these extreme load conditions, even when applied for a limited number of loading cycles. Consequently, in the initial studies, consideration was given to the low-cycle fatigue resistance of these joints. The design was typically a jacket-type multi-legged trussed tower with tubular members. Joints normally consisted of 42 to 48 inch diameter column members and tubular brace members of approximately 16 to 24 inches in diameter. The design of these joints was typically based on extreme load conditions which would produce a nominal stress reversal in the diagonal members of 28 ksi.

These studies readily indicated that the flexibility of the column member wall was the main source of stress concentrations and consequently resulted in early fatigue failure; "hot spot" stresses were identified. In order to stiffen the column wall, one considered increasing the column wall thickness, using external ring stiffners or in-plane gusset plates, or possibly, overlapping and interwelding the web members.(5,6,7,8,9) Comparative studies clearly indicated that a joint with a thickened column wall and non-overlapping web or branch members was superior from the point of view of strength and low-cycle fatigue resistance. These initial studies successfully guided the design of tubular joints under these particular environmental conditions (extreme waves). However, these design rules became inadequate where offshore developments moved into more hostile regions. Not only the extreme sea state, but also the repeated cyclic low-forcing wave effects became significant design considerations. Hence, a more general approach to fatigue and cumulative fatigue damage became necessary.

Comparative studies of the type noted before could no longer provide sufficient input to determine the structural response resulting from the broad load spectrum which now had to be considered in design. Consequently, other theories and their applicability had to be evaluated. Miner's linear cumulative fatigue damage hypothesis was used in establishing the effect of cumulative fatigue damage. Equally important, however, was the need of determining the stress versus number-of-cycles, or "S-N", curves necessary to determine the ultimate fatigue resistance for given stress levels. Considering the complex environmental conditions and material response, different theories have been considered and their application to tubular joint design evaluated. In this respect, a linear-elastic fracture mechanics approach to determine theoretical S-N curves has been found to yield a satisfactory correlation between theoretically predicted and experimentally observed fatigue crack behavior.(10)

In principle, this type of approach is needed in order to develop design criteria for the many different systems which are designed to operate in the often hostile offshore regions now consid-

ered for development. The severity of the environmental loads requires a new approach to tubular joint design considering both joint strength, joint stiffness and fatigue reisitance. There is no doubt that solutions can be obtained. However, it will require the most advanced integrated approach of loading methodologies, computer modeling techniques, analysis, fabrication and material science. Not only material properties affecting the ultimate response are significant, but also the prediction of the highly complex dynamic response of offshore structures, both stationery and floating, steel or concrete. In that instance, the combined effects of waves, currents, winds, soil conditions and earthquake loadings should be fully considered. Only a highly advanced approach in analyzing the anticipated performance will provide the basis for a satisfactory design.

I noted in the last paragraph, concrete structures as well. While I did address myself to the fatigue aspect of tubular joints and the overall design requirements of steel platforms, concrete or pre-stressed concrete structures require an equally rigorous design approach. Consequently, the engineer's total understanding of the many environmental and structural performance aspects in every phase of design, is essential. Extending knowledge and performance data to structures similar in concept, but located in different environments, does require the utmost of engineering attention in order to prevent the potentially catastrophic outcome of extending the "present state-of-the-art" to new regions without full consideration of all factors involved. In that respect, I hope that the subsequent discussions and future contributions published by the IABSE on this subject may aid the profession in a field which , because of the short history and engineering complexity, requires our fullest attention.

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Ultimate Strength Design Formulae for Simple Tubular Joints

Formules du calcul à la résistance limite pour les noeuds simples de profilés circulaires

Formeln für die Ermittlung der Traglast von einfachen Knotenpunkten in Rohrprofilen

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1. INTRODUCTION

One of the Committees in the Architectural Institute of Japan is carrying out revision of the "Specification for Design of Tubular Structures in Steel" that was first published in 1962. Although the Specification is applicable to the building structure, it may provide a good deal of information for the design of offshore structures of tubular members.

This report discusses about the experimental grounds of those provisions for the design of the tubular X, T, Y and K-joints for static loadings which are presently under deliberation in the Committee. The provisions are based on the ultimate strength formulae that were selected for such use from the results of a series of regression analyses of the test data obtained in Japan and the U.S.A. since 1963.* The derivation of the formulae is described in detail in Reference [1].

All the existing ultimate strength formulae for these joints have been subjected to two questions as follows:

1. Most of the existing ultimate strength formulae tend to overemphasize the strength of the T, Y and K-joints when the diameter to thickness ratio of the chord (D/T) becomes greater than about 50.
2. The strength of the K-joint increases as the two braces intersect and then overlap with each other. This behavior is not adequately taken into account in the existing formulae.

To overcome these difficulties the reanalyses of the test data were carried out as a continuation of the past studies by the authors and their colleagues[2],[3].

The strength of the tubular joints under static loads is an influencing factor in determining the design of any tubular structure and yet it still covers some areas that are not fully understood at the present stage, which may be clear from the later discussions in this report. In this regards the authors wish to welcome any comment on the proposals presented in this report.

2. DEFINITION OF ULTIMATE STRENGTH

The ultimate strength referred herein is the maximum axial compressive force applied at the brace ends when a joint fails as a result of excessive local bending deflections of the chord walls. The strength of a joint that fails owing to

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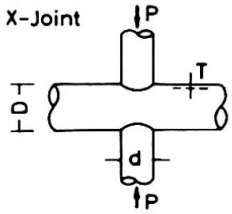
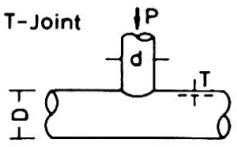
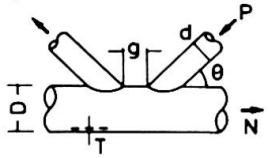
* Additional test data are now being gathered from Europe through the activity of the Subcommission XV-E of the IIW.

failures in a member, such as fracture of the tension brace or local instability of the compression brace, is outside the scope of the present definition of the ultimate strength of the joint.

Table 1 Proposed Ultimate Strength Formulae

Type of Joints	Predicted Ultimate Strength, P_u
X-Joint 	$P_u = \frac{6.57}{1 - 0.810\, d/D}\,\sigma_y T^2$ (a)
T-Joint 	$P_u = 6.43\,[1 + 4.60\,(\frac{d}{D})^2]\,\sigma_y T^2$ (b)
T, Y and K-Joints  $\bar{N} = N/\sigma_y A$, $A = \pi(D-T)T$	$P_u = 2.11(1 + 12.1\frac{d}{D})\, f_g\, f_\theta\, f_{\bar{n}}\, \sigma_y T^2$ (c) $f_g = 1 + 3.88(1 - 20.9\frac{T}{D})(1 - 0.530\frac{d}{D}) \cdot [1 + \frac{2}{\pi}\tan^{-1}(0.237 - 0.183\frac{g}{T})]$, but not less than 1.0 $f_\theta = (1 - 0.167\cos\theta + 0.049\cos^2\theta)/\sin\theta$ $f_{\bar{n}} = 1 + 0.262\,\bar{N} - 0.391\,\bar{N}^2$ ($\bar{N}$: positive for tension)

The local failure of the chord walls occurs also at the points where tension braces are attached. The final rupture of these joints is controlled by cracking of the chord and/or brace walls at the toes of the brace to chord welds. The joints that fail in this manner always attain a far greater strength than that in the former case where the braces are under compression. The ultimate strength data for the joints under tension should therefore be treated separately and are excluded from the regression analyses in Referenc [1].

According to the past tests, most tubular joints reached the maximum load after full plastic deflections of the chord walls were produced at the local portions where the braces were attached, and then unloading took place. A typical load-deflection curve of such joints is shown by the curve 1 in Fig. 1. In some joints, however, the overall stiffness increased again after they sustained full plastic deformations of the chord walls and eventually carried a greater load than the first maximum load. The load-deflection curves of the latter type are shown by the curves 2 and 3 in Fig. 1. The ultimate strengths used for the analyses were the first maximum loads that were attained by the joints after sufficient areas of the chord walls yielded.

K-joints often fail in a combination of various failure modes depending upon the dimensions of the joints. Even though the final failure of a K-joint was governed by one of the other modes than the excessive local deflections of the chord walls, the ultimate strength of the joint was included in the data so far as the joint sustained full plastic deflections of the chord walls under the compression brace at the maximum load.

3. PROCEDURE OF REGRESSION ANALYSIS

The multiple regression analyses were carried out to develop the best-fit equations of the ultimate strengths of the joints. In the process of building a mathematical model for a prediction equation, it was first assumed that the joint was able to be replaced by a simple and fictious structure of which ultimate strength would represent the ultimate strength of the actual joint.

Such a simplified model structure is a ring with an effective width Be. The ring has the same diameter D, thickness T and yield stress σ_y as those of the actual joint and is subjected to concentrated forces acting at d distant points, where d is the outer diameter of the brace. These concentrated forces represent the axial compressive force P in the brace.

According to the simple plastic theory, the collapse load of the ring Pu is given by the equation,

$$Pu = \frac{Be}{a} f_0(\frac{d}{D}) \sigma_y T^2 \qquad (1)$$

where a is the mean radius of the chord. Be and $f_0(d/D)$ are functions of geometrical parameters of the joint and vary with the type of the joint.

Therefore, the model may be written in the form,

$$Pu = f_0(\frac{d}{D}) f_1(\frac{d}{D}, \frac{T}{D}, \frac{g}{T}, \theta, \bar{N}) \sigma_y T^2 \varepsilon \qquad (2)$$

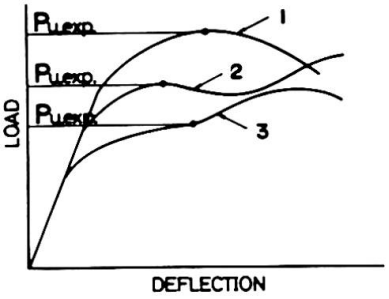


Fig. 1 Examples of Load-Deflection Curves

in which Pu is the ultimate strength of the joint, g is the clear space (gap) between the two braces, θ is the angle of intersection between the compression brace and the chord and $\bar{N}$ is the dimensionless axial stress in the chord (See Table 1). ε is the error term, which was assumed to be multiplicative rather than additive because the model (2) consists of multiplicative terms of the influencing factors each of which has a certain physical meaning [1]. It is assumed here that errors ln(ε) are independent random variables with mean zero.

Since the postulated model was nonlinear in the parameters, the linearization and iterative techniques were exercised to fit the model by the method of least squares [4]. A series of such analyses were performed with several alternative models for the functions f_0 and f_1 of Eq. 2. The resultant regression equations were compared on the basis of the "multiple correlation coefficient R^2". The final selection of an equation was made such that the selected equation would explain the variation of the ultimate strength data better (attain a larger R^2) with less predicting variables.

In Reference [1] are shown all the data used for the analyses and also the reference sources of them. An effort was made to utilize as far as possible measured values rather than nominal values for the independent variables. Although the yield stress in the circumferential direction may be more meaningful in this model, such yield stress is not usually measured in most experimental works. σ_y,s adopted herein are the longitudinal yield stresses measured on tensile coupons cut from the as-rolled chord materials.

4. RESULTS OF ANALYSES AND DISCUSSIONS

The three equations shown in Table 1 were selected as the ultimate strength prediction equations for the tubular X, T, Y and K-joints. Eq. c is applicable to any of the T, Y and K-joints, where g is infinitely large in the T and Y-joints and becomes negative when the braces overlap in the K-joint. R^2 was of 92%, 95% and 91% in Eqs. a, b and c, respectively.

The residuals provided by Eq. c are plotted overall in a form of a frequency histogram (Fig. 2). It appears that the residuals follow a normal distribution. According to a chi-square goodness of fit test this assumption of normality was found to be acceptable at the 0.05 level of significance. From this it may not be unreasonable to assume that, if the model is correct, errors ln(ε) are normally distributed.

Since the models are nonlinear, statistical tests that are true for the linear case do not apply. However, since the number of observations n is large, the 95% confidence limits for an

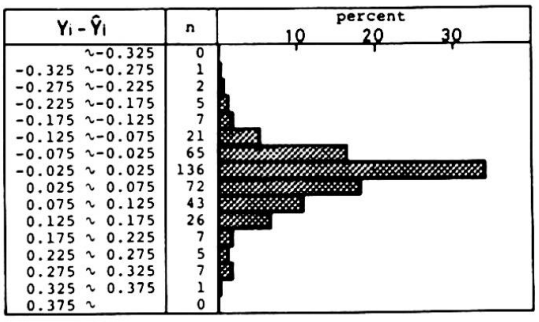


Fig. 2 Histogram of Residuals

individual predicted value are approximately given by

$$Pu \cdot e^{\pm 2s} \qquad (3)$$

where s is an estimate of standard deviation and is approximately obtained by the equation,

$$s = \sqrt{\frac{\text{(residual sum of squares)}}{n - p}}$$

in which p is the number of parameters in the regression equation. The above statement is valid only when the errors $\ln(\varepsilon)$ are normally distributed.

The predicted mean value Pu and the approximate 95% confidence limits according to (3) are compared with the test results "Pu,exp"in Figs. 3 through 6. These plots indicate no strong abnormality in the residuals and the present regression analyses would not appear to be invalidated.

It is important to note that the formulae in Table 1 are applicable only within the ranges of variation of the predicting variables. Figs. 8, 9 and 10 illustrate how the predicting variables varied in the test data.

The Japanese Specification referred to earlier tentatively assumes a factor of safety of 2 on the predicted ultimate strengths of the X, T, Y and K-joints. The allowable force may be increased 50% above Pu/2 when the joint is under combined permanent and temporary loadings. This safety factor appeared to be conservative from Figs. 3 through 6. In order to calculate a probability of failure for a joint, however, it is necessary to know probability distributions of loads and the yield stress of the materials.

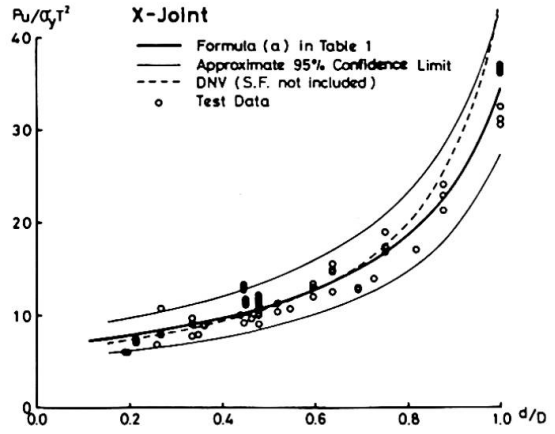


Fig. 3 Predicted Ultimate Strengths and Test Results

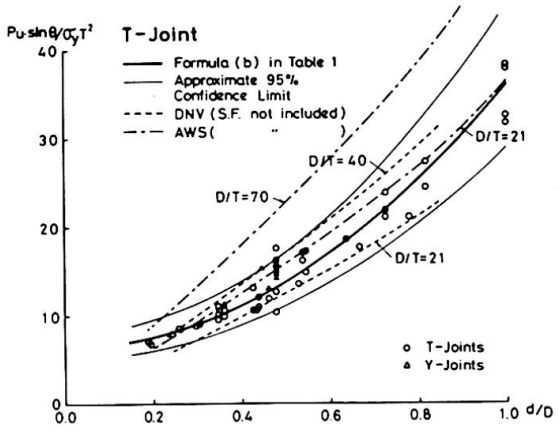


Fig. 4 Predicted Ultimate Strengths and Test results for T and Y-Joints

5. COMPARISON OF PROPOSED FORMULAE WITH EXISTING FORMULAE

The proposed formulae and the test results are compared with the formulae recommended in the AWS and DNV-Codes [5],[6] in Figs. 3 through 6 where factors of safety are not taken into account. The DNV-formula agrees well with Eq. a as well as with the test results for X-joints. However, both the AWS and DNV-formulae are not necessarily consistent with the formulae b and c nor with the test results for T, Y and

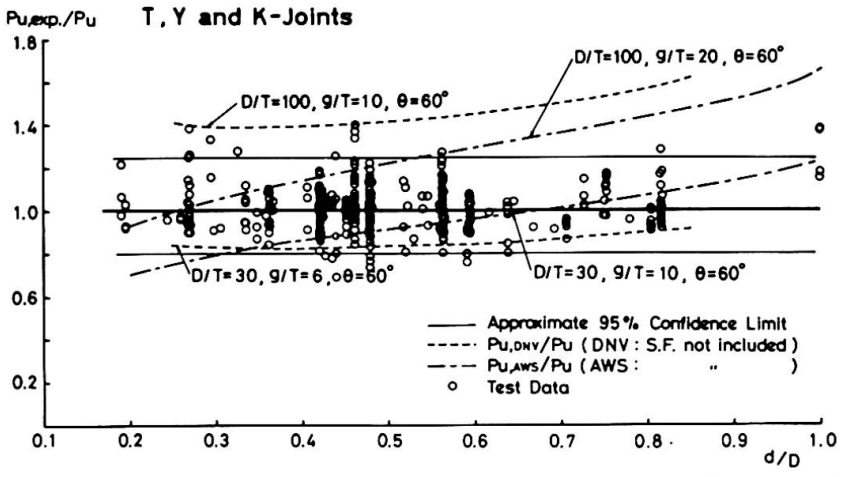


Fig. 5 Ultimate Strengths of T, Y and K-Joints: Comparisons between Formula c, AWS and DNV Formulae and Test Data

k-joints. In most cases, the AWS and DNV-formulae are risky when D/T=100 and are too conservative when g/T is less than about 8. Response of the AWS formula to a variable d/D looks to be different from what is observed in the test results for K-joints. Both the formulae are generally applicable to the T and Y-joints and the K-joints with extended braces, when D/T is less than 40.

Another comparison is made between Eq. c and the formula by Okumura et al [7] in Fig. 7. The ultimate strengths of T, Y and K-joints predicted by the Okumura's formula are scattered between the two dashed lines in the figure (when θ=60°). This formula does not overestimate the strength of these joints with large D/T ratios, but it again is too conservative for a majority of K-joints with intersecting braces.

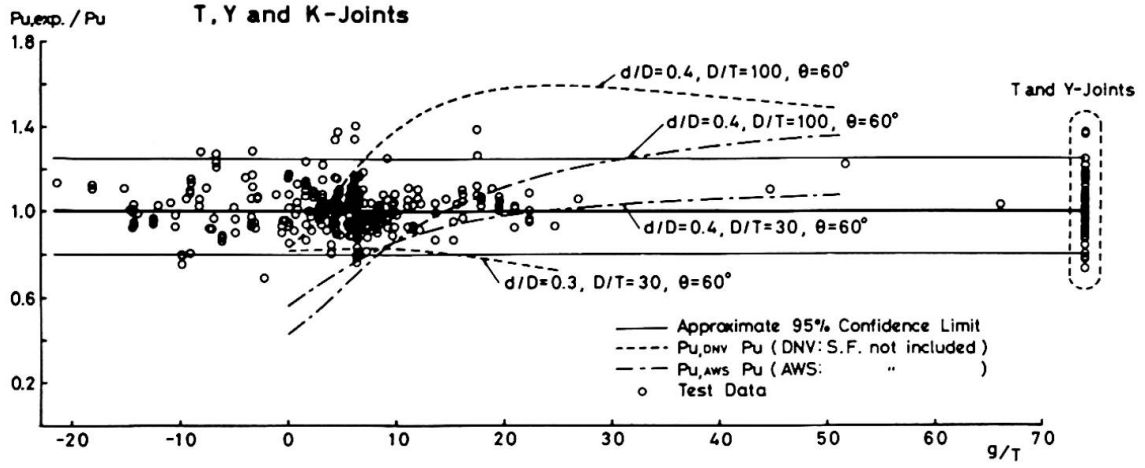


Fig. 6 Comparisons between Formula c, AWS and DNV Formulae and Test Data

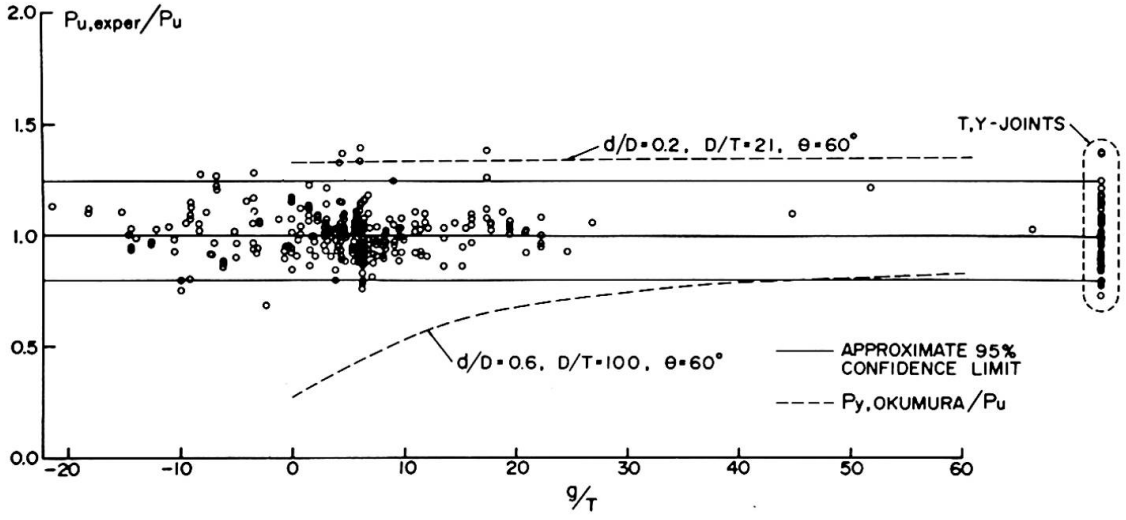
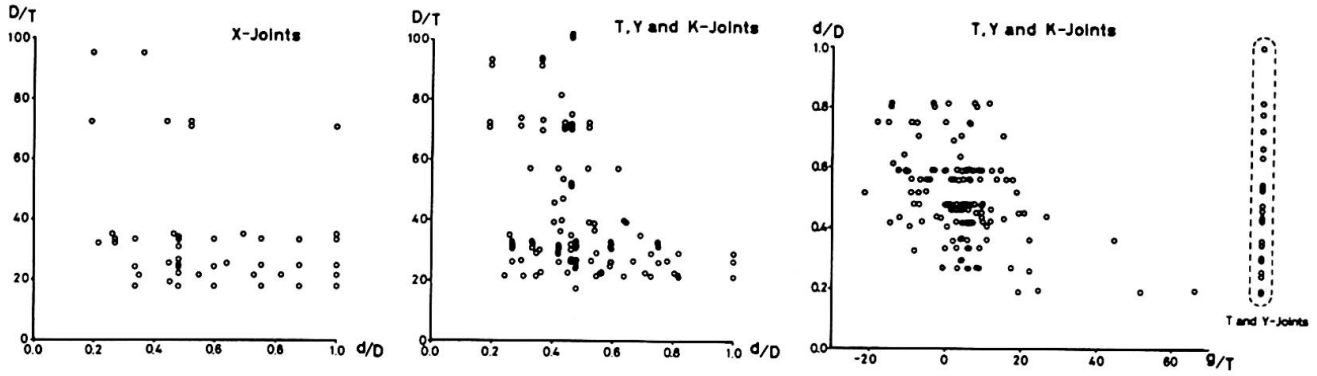


Fig. 7 Formula by Okumura et al Compared with Test Results (T, Y and K-Joints)



(from left) Figs. 8, 9, 10 Independent Variables in Test Data

6. CONCLUSIONS

The ultimate strength formulae summarized in Table 1 is applicable to the tubular X, T, Y and K-joints with a wide range of variation of each of the geometrical parameters. The formula c in the table can also applies to the K-joints with overlapping braces, which may be the first of such examples.

The test data, however, are still unavailable for some important ranges of variation of predicting variables. Examples of the areas that require further studies are:

1. The joints with a very heavy chord ($D/T<20$) and with a very light chord ($D/T>100$). The joints in these two extremities are often used in Jack-up and semi-submersible type offshore rigs, respectively.
2. The K-joint with large braces ($d/D\simeq1$).
3. Resistance of the joints under bending at the brace ends. It is to be noted that in the regression analyses an effect of secondary bending moments on the strength of the K-joint was treated merely as a factor that induces random errors.
4. The joints in high strength steels. The effects of material properties and heat treatments are still unaccountable factors that require additional work. Most of the test results were obtained through the joints made of cold formed tubes in mild steels or in low alloy medium strength steels.

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SUMMARY

This report presents the design formulae for the tubular X, T, Y and K-joints under static loads. It also discusses about the experimental grounds on which the formulae are based. The proposed formulae are compared with various existing formulae.

RESUME

Ce rapport présente les formules du calcul pour les noeuds de profilés circulaires, en forme de X, T, Y et K sous l'influence des charges statiques. Il présente aussi les bases expérimentales qui ont permis l'établissement de ces formules. Les formules proposées sont comparées avec d'autre formules existantes.

ZUSAMMENFASSUNG

Dieser Bericht enthält Bemessungsformeln für X, T, Y und K-Knoten von Hohlprofilen, unter statischen Belastungen. Die den Formeln zugrundeliegenden experimentellen Daten werden angegeben. Die Formeln werden schliesslich mit verschiedenen bereits bekannten Formeln verglichen.

Méthode de calcul à la fatigue

Methode zur Bemessung auf Ermüdung

Fatigue Design Method

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1 - INTRODUCTION

Deux communications du rapport préliminaire faites par nos confrères japonais donnent des résultats très intéressants sur la tenue à la fatigue des assemblages tubulaires en milieu marin.
Les développements présentés concernent des études expérimentales sur des noeuds types.
Cette communication a pour but de montrer les problèmes particuliers posés pour le dimensionnement des structures off-shore par l'utilisation des éléments précédents, à la fois sur le plan de la note de calcul et sur le plan pratique de la conception de la charpente.

2 - METHODES DE CALCUL

Le principe de l'étude à la fatigue consiste à déterminer les variations σ de contraintes élastiques en un point donné pour différents états de mer classés suivant la hauteur H des vagues mesurées de crête à creux. Il est possible de déterminer la fonction :

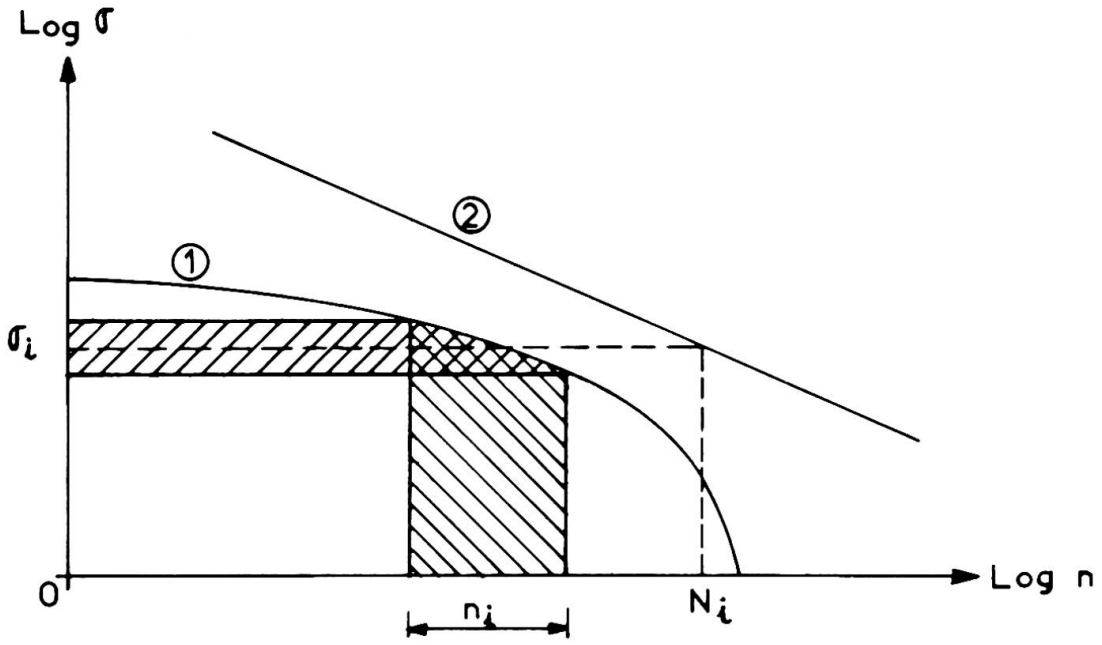
$$\sigma = f(N) \qquad N \text{ nombre de cycles}$$

En divisant les contraintes en q classes, de valeur moyenne σi, le critère de PALM GREEN MINER s'écrit :

$$\sum_{i=I}^{q} \frac{ni}{Ni} \leq I$$

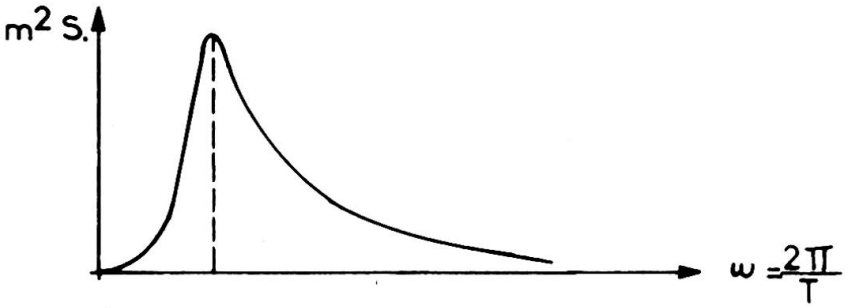
N_i correspondant au nombre de cycles maximum que peut supporter la structure pour des variations σi de contraintes.

L'utilisation du critère décrit ci-dessus pose quelques problèmes d'application pratique que nous détaillons.

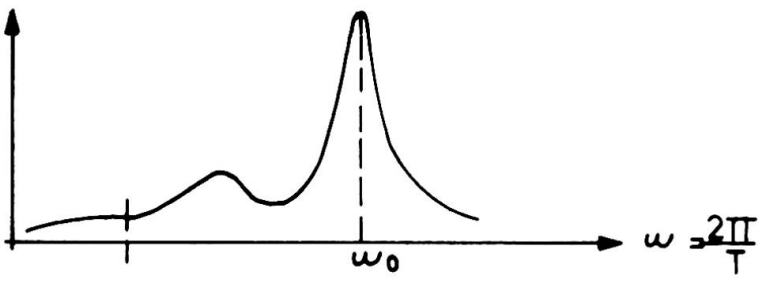


① Distribution des variations de contraintes

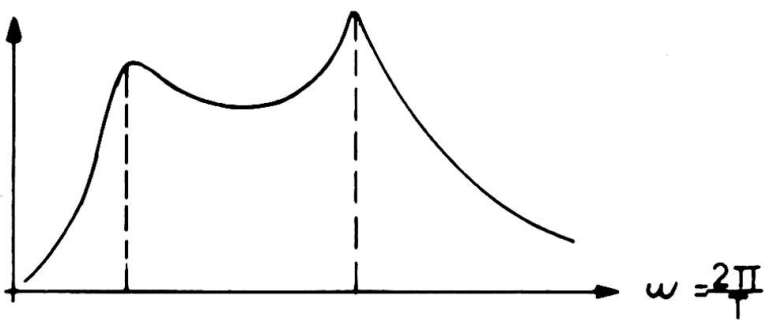
② Courbe de fatigue



Spectre d'énergie
(S)



Fonction transfert
(F)



Spectre de réponse
(Sr)

2.1 - Parmi les classes de vagues, celles correspondant aux hauteurs les plus faibles (1 à 3 mètres) contribuent pour une part très importante au dommage. En effet, la contrainte dynamique qui en résulte est très importante par rapport à l'effort statique. Ce phénomène résulte de la période qui est en général voisine du mode propre de vibration de la structure. Une telle analyse conduit à une estimation trop pessimiste du dommage.

Pour remédier à cette difficulté on peut faire une analyse spectrale des hauteurs de vagues (utilisation de spectres comme ceux de JONSAP ou de PIERSON MOSKOWITZ).
L'étude du comportement dynamique de la structure pour une direction de vague donne une fonction transfert F.
On obtient le spectre de réponse au niveau des contraintes grâce à l'équation :

$$(S_r) = (F^2)\ S$$

A chaque spectre S_i on peut associer la valeur :

mo_i moyenne $= \int_o^{+\infty} {}^i S_r\ (\omega)\ d\omega$ et la probabilité P_i de dépassement de σ est égale à : $P_i = \exp.\ (-\ \frac{\sigma^2}{mo_i})$

Il en résulte que si l'on tient compte de tous les spectres, chacun ayant une probabilité P_i, on arrive à :

$$P = \Sigma_i\ p_i\ P_i$$

Cette fonction permet la détermination de $\sigma = f(N)$

Une telle démarche conduit à une évaluation plus optimiste (dommage cumulé de 30 % inférieur environ).
L'effet dynamique d'amplification diminue dans la mesure où les pointes du spectre et de la fonction transfert sont très éloignées.

2.2 - Un autre problème est celui de l'analyse des différentes phases de la vie de la structure. Celle-ci peut être remorquée (voir figure 1) puis basculée et mise en position.

Deux dommages peuvent être évalués :

- celui en cours de remorquage, lié à la route maritime choisie et à l'époque de l'année (été ou hiver)
- celui pendant la vie de l'ouvrage

Le cumul de ces deux valeurs est très discutable car les schémas hydrostatiques d'étude sont très différents. Actuellement, il n'existe pas de théorie valable sur l'accumulation des dommages dans des conditions très variables de temps.

Heureusement, les zones affectées par les efforts maxima sont très différentes suivant les périodes considérées.
Dans un "Jacket" (figure 2), les efforts en cours de basculement dépendent en grande partie des zones ballastables et affectent peu les parties hautes et basses, très sensibles en phase définitive.

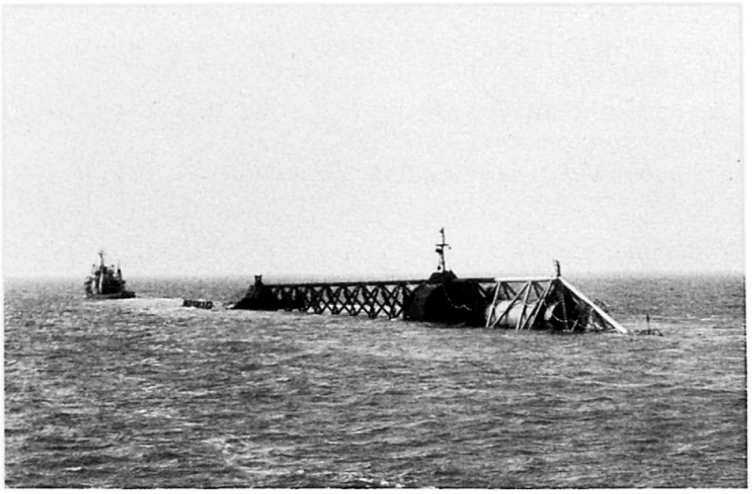


Figure 1 - Colonne articulée en cours de remorquage

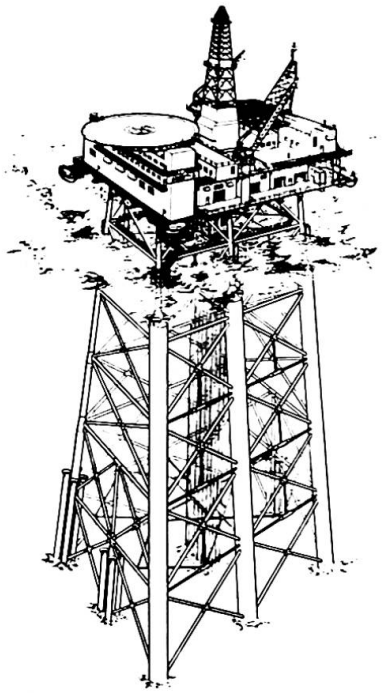


Figure 2 - Plateforme BRENT A

3 - MODIFICATIONS PERMETTANT DE REDUIRE LES DOMMAGES DUS A LA FATIGUE

Les calculs de fatigue nécessitent une bonne connaissance des conditions météorologiques et de la structure. Une telle coïncidence ne se produit qu'en fin de chantier d'assemblage. Il est donc utile de procéder à des modifications ne mettant pas en cause l'ensemble de la construction sous peine de retards inadmissibles.

Ces modifications possibles sont d'ordre très différent :

3.1 - Les efforts en cours de remorquage proviennent d'une statistique de houle liée à la période pendant laquelle se fait l'opération et au choix de l'itinéraire.

Il est possible de réduire le dommage en étudiant des itinéraires longeant les côtes avec des abris préalablement aménagés (coffres d'amarrage et signalisation, préinstallés). Un risque important réside dans les prévisions météorologiques à plus de 24 Heures en cas d'opération en pleine mer.

Une colonne articulée (voir figure 1) a pu être mise en place en Mer du nord, notamment sur le champ de BRENT grâce à une telle étude.

La position de remorquage peut constituer également un facteur important pour diminuer le dommage ; En restant dans le cas particulier des colonnes articulées, le basculement préalable en site abrité et un remorquage en position verticale, donnent une solution intéressante. Dans un tel cas, la colonne pilonne très peu même sous l'action de vagues de 3 à 4 m. Il est donc possible d'effectuer un remorquage pendant l'hiver en Mer du Nord.

Rappelons qu'une telle possibilité a été utilisée pour la mise en place de la colonne articulée sur le champ de FRIGG et pour celle installée sur le champ exploité par MOBIL.

L'étude des risques de fatigue est remplacée par une reconnaissance très soignée des hauts fonds sur l'itinéraire de remorquage.

3.2 - Les structures marines off-shore peuvent appartenir :

- soit à des ensembles composés de tubes (voir figure 3)
- soit à des ensembles composés de poutres en caisson (Pont de plate-forme fixe - voir figure 4)

Dans le premier cas, les risques de fissuration de fatigue sont concentrés dans les régions voisines des noeuds. Pour réduire les pointes de contraintes génératrices de fatigue, il est possible de mettre en place des diaphragmes internes lorsque le diamètre des pièces le permet. Ces diaphragmes doivent avoir des tracés très soignés pour obtenir une bonne égalisation des efforts.

L'étude peut être faite grâce à une modélisation convenable du noeud et à l'emploi d'un programme aux éléments finis. Les dissymétries du noeud et des barres adjacentes ainsi que les cas de charges à envisager, limitent les analyses. Bien souvent, les diagrammes obtenus permettent des conclusions qualitatives et non quantitatives.

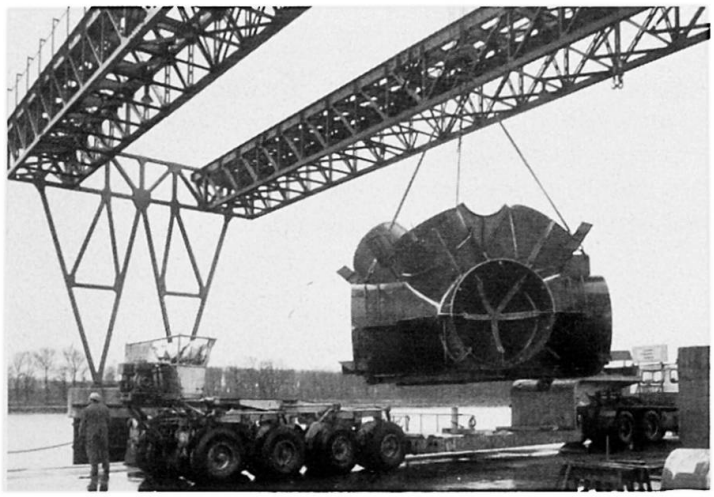


figure 3 - Noeud de plate-forme Penta 87

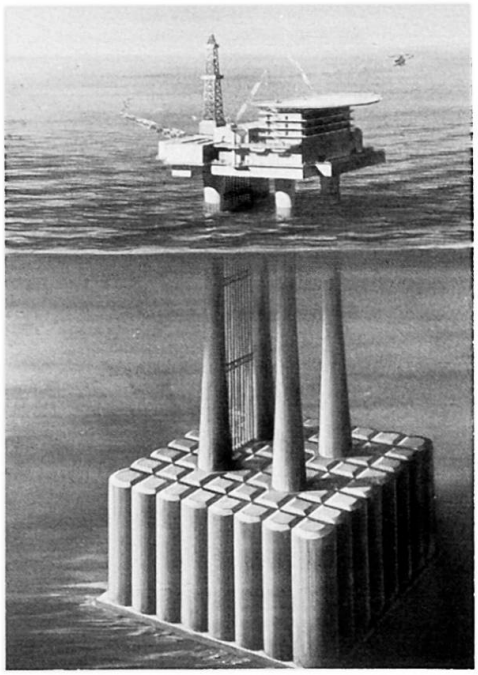


figure 4 - Cormorant
Pont en acier

Pour des barres de petit diamètre (Ø < 1 mètre), les aménagements sont très limités si le noeud est complètement réalisé. Dans le cas où les barres composées de tubes sont libres aux extrêmités, on peut limiter les risques d'arrachement grâce à des recouvrements (over lapping). Sinon, la solution "gousset" ou "anneau de renfort" peut réduire légèrement les efforts longitudinaux, mais n'apportent pas de réduction à la contrainte d'arrachement (hot spot stress).

Dans le deuxième cas (poutres caissons), les efforts de fatigue sont sensibles au droit des ouvertures (diaphragmes ou passages de canalisation au droit des âmes de caissons). Celles-ci sont à border par des goussets suffisants.

La complexité des efforts transmis, nécessite une étude très poussée en éléments finis et il convient d'éviter des sections horizontales de tôles trop sollicitées dans les calculs de prédimensionnement. Enfin, les efforts dynamiques dus à la houle peuvent entraîner des renforts localisés sous les supports des modules.

RESUME

Le comportement en fatigue des structures marines nécessite une étude difficile qui doit tenir compte des spectres de houle et du comportement dynamique du système. Les risques de dommage à prendre en compte en service ou en phase de remorquage peuvent être diminués grâce à un certain nombre de dispositions constructives ou d'aménagements des itinéraires de remorquage.

ZUSAMMENFASSUNG

Das Ermüdungsverhalten von Bauwerken im Meer erfordert aufwendige Untersuchungen, welche das Wellenspektrum und das dynamische Verhalten des Systems berücksichtigen müssen. Die in Betracht zu ziehenden Risiken einer Beschädigung im Betrieb und beim Abschleppen können dank einer Anzahl von baulichen Massnahmen oder durch sachgemässe Auswahl der Schleppwege vermindert werden.

SUMMARY

Fatigue behaviour of off-shore constructions requires complex investigations which must allow for both swell spectra and overall dynamic behaviour. The damage risks to be reckoned with under service conditions and during towing can be reduced by resorting to a number of engineering solutions or by determining suitable tow routes.

A new Kind of Hybrid Construction

Une construction hybride nouvelle

Eine neue hybride Konstruktion

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1. Introduction

In the earlier stages of the development of deep water oil production platforms for the North Sea the choice was oversimplified to one of using a pile supported steel jacket or a concrete gravity structure. Recent designs have combined both materials and are often referred to as "hybrids". (Fig. 1). In this paper the arguments for a new type of hybrid are explored and the resulting design (Tricon) briefly described. (Fig. 2).

2. The Problems and Some Solutions

An offshore platform structure in deep and exposed waters is a special case of a tall building with almost its entire payload concentrated in the "penthouse", subject to exceptionally large horizontal loadings of a cyclic nature and with uniquely difficult site access and foundation conditions. However, having accepted these rather special design and construction parameters, the design of such a structure should still be tackled and ultimately make as much sense as that of any other structure. In other words, the design concept should be governed by the "payload" and the permanent working conditions, be reasonably easy to construct and in the process possibly permitting some temporary overstressing.

The present generation of self-floating "jacket" structures does make sense in that respect, albeit with some obvious shortcomings.

The tubular joints, the node points, have become so large that the secondary stresses have reached a magnitude of prime importance, requiring a great amount of internal stiffening. They are however still not large enough to permit more than one or two welders to work inside the joint. Furthermore, the number of node points in a major structure is still so large that the prefabrication of these node points is a very critical element in the construction programme.

A tubular space frame with a minimum number of members and hence node points leads to a much more satisfactory structural concept. The tubular members become so big that they achieve a natural buoyancy even if made of reinforced or prestressed concrete, and the node point become so spacious, that a considerable work force can get inside them They also become so accessible, that the internal stiffening members can be designed and positioned so correctly that secondary stresses can be reduced to what they ought to be; i.e. of secondary importance.

Another serious shortcoming of the larger pile supported jacket structures is the great proportion of weather and crane barge dependant offshore work. This proportion will ultimately, with increasing water depths and wave heights and narrowing weather windows, reach a "point of no return", and in the North Sea it lead to the introduction of the "gravity" structures.

The present generation of concrete, steel or hybrid gravity structures has solved the offshore installation problem by avoiding it - but at a very heavy premium. For their construction they require very special facilities - such as extremely deep and sheltered water - and their design leave no room for the designer to produce an optimum solution to the prime purpose of the structure; i.e. to support the payload safely and economically above the waves. It is a fact, that the optimum solution for the permanent conditions is incompatible with the temporary conditions, and that the structure is governed by the requirements for floating stability and structural strength during the towing and sinking operation. Having met these temporary conditions as well as those imposed by the restrictions of the available construction site, the designer is then left with the rather unsatisfying task of just checking, whether the structure is adequate for the permanent conditions. Hardly the way to produce the most economic structure, and at times, when the upper soil strata of the seabed are too weak, not even a way to produce just an acceptable structure.

It is in the approach to the foundation problem that both types of existing offshore structures show their worst limitations.

The pile supported structures suffer not only because the piles are difficult and costly to drive, but also because they are not the best and most economic foundation method. Loads of the same order of magnitude are often supported under water; bridge piers in connection with river or harbour crossings. In such cases one would generally dig down to a suitable foundation level and one would never contemplate driving piles to carrying capacities of several thousand tons a piece.

Similarly the gravity structures suffer not only because they become disproportionately large and impose high stresses on the upper and weaker soil strata, but also because they impose an impossible tas

on the designers. It is impossible to tailor make a foundation to suit an unprepared and practically unknown seabed or to determine the right length and strength of a penetrating skirt without exact information on the soil into which the skirt has to penetrate. Still, this is the very first design decision, which has to be made.

From the above considerations it would appear, that deep footings (assuming it is feasible to install them) are cheaper than long steel piles and also a more rational foundation method than resting directly on top of the seabed.

They are independant of the seabed topography and they eliminate the risk of sliding and the risk associated with scour of the seabed. They can furthermore be designed properly, taking full account of the prevailing soil properties, if the construction of the footings can be reversed to become the last item and not the first on the construction programme.

3. The Design

The above design considerations have resulted in the Tricon concept. Tricon consists of three basic elements; (i) on the top a short legged jack-up platform designed to the clients requirements, (ii) a main three legged space frame optimised to carry the payload and resist wave action, made up of large tubular members with few node points, and (iii) caisson footings sunk to a firm foundation.

The tubular space frame is constructed horizontally as a normal jacket structure. All the tubular members have considerable natural buoyancy and the structure as a whole floats in a very shallow water. The footings are circular hollow cylinders suitably stiffened to be able to spread the column load over the required foundation area. They can be made of steel, but even made of concrete will they possess enough buoyancy to float in shallow water.

Since the structure is being constructed horizontally, (Fig. 3) the footings can be added as the last item on the programme, and even with the diameter fixed the height can be increased until the last minute.

The structure is obviously extremely stable floating horizontally, but because of the positive buoyancy of all the tubular members it remains stable during all phases of the upending process which is achieved by controlled flooding.

Having touched down with the three footings resting on top of the seabed the tops of the columns are well above the waters surface. The caissons are now sunk into the seabed by means of excavation inside the footings. The excavation process is a self-contained and

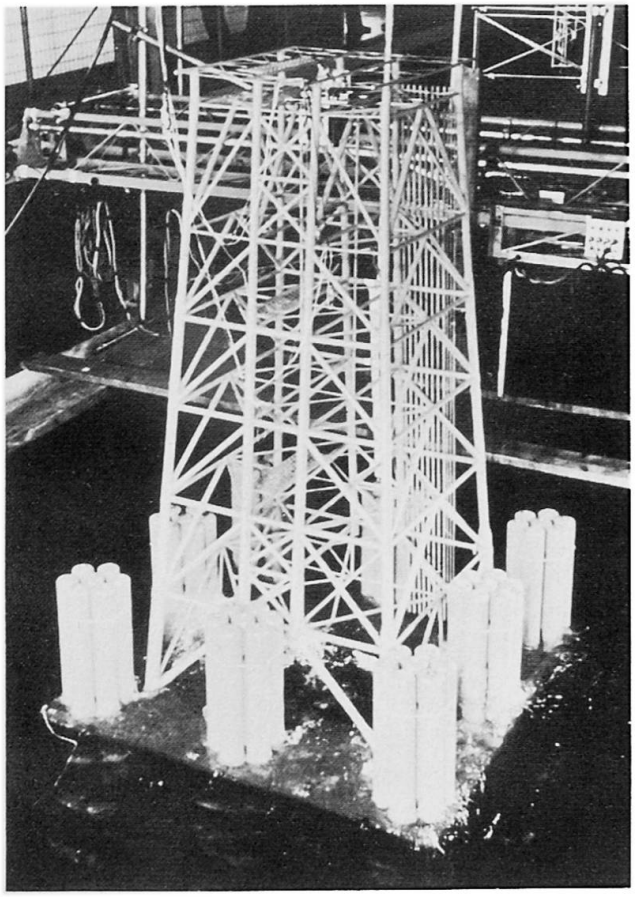


Fig. 1. RDL (NORTH SEA) HYBRID

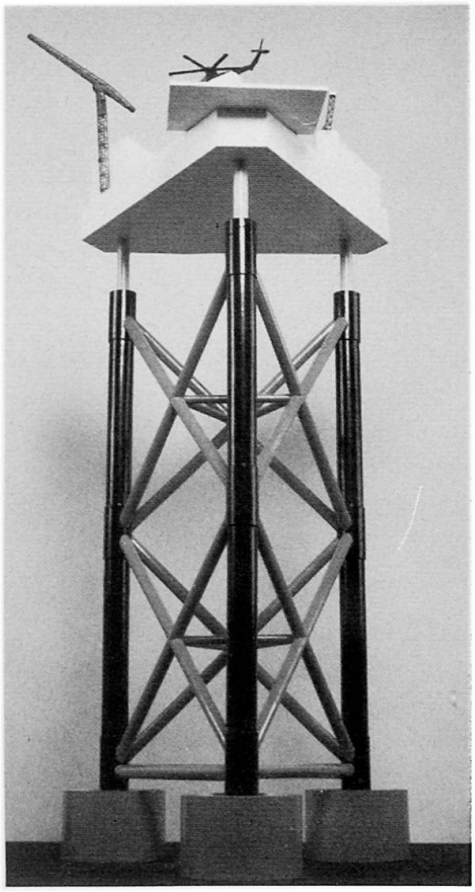
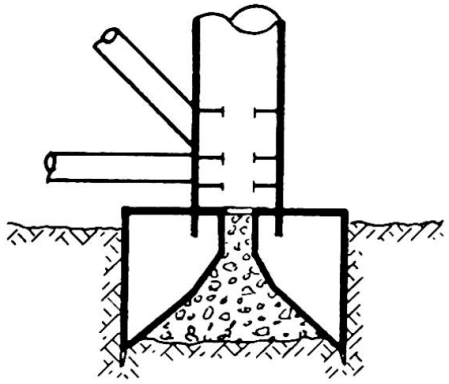


Fig 2. TRICON

Fig 3. TRICON IS FABRICATED & FLOATED HORIZONTALLY

practically weather independant operation. The necessary tools are built into the structure and there is plenty of room in the top of the columns to accommodate the working crew.

The excavation takes place under water with purpose made digging or tunnelling tools dragging the excavated material from the perimeter of the footings to the centre, where it can be removed by pumping or air lifting.



Caisson Foundation

As soon as the cutting edge of the footings has penetrated into the seabed and sealed off the interior of the footings from the outside water pressure, the structure gains instant "fixity" and it is possible to reduce the internal pressure and thus utilising the external hydrostatic pressure to assist the penetration into the seabed. In that respect the sinking of the Tricon feet is easier and more controllable than a caisson sinking in compressed air or even an open excavation for a bridge pier. Compared to the piledriving, where each hammer blow must exert a force of several thousand tons to achieve penetration, the excavation process is obviously much simpler, requiring a force of only a few tons to break down the soil into removable lumps and thus achieve penetration into the seabed.

The excavation process is continued until the tops of the columns are at a predetermined depth below the waters surface, access to the working space in the columns being gained through removable access shafts. With the footings at the right depth the space underneath the footings is backfilled with granular material and the Tricon foundation is completed.

The temporary access shafts are then removed and the underwater foundation is ready to receive the "jack-up" superstructure.

To land the "jack-up" legs on the Tricon columns requires a spell of calm weather, but only a very short spell, and no calmer than what is required to carry out one lift with a crane barge; i.e. equivalent to bringing one pile section into the pile guides or placing the hammer on top of one pile.

If required the whole process can be put in reverse for the removal and movement to a similar site. Excess hydraulic pressure can be utilised to force the raising of the caissons in a controlled manner.

If, as intended, the Tricon concept combines the structural economy of a "jacket" with the offshore installation economy of a gravity structure, it should logically lead to cheaper offshore structures.

SUMMARY

The new type of hybrid construction "TRICON" divides naturally into three distinct main sections:

the top - a conventional three-legged "jack-up" platform

the tower - a triangular tubular space frame with minimum number of members

the foundation - three deep caisson footings sunk into the seabed to a suitable foundation level.

It combines the best structural features of a "jacket" and the offshore installation advantages of "jack-ups" and "gravity" structures.

RESUME

La nouvelle construction hybride "TRICON" se divise en trois parties distinctes:

le sommet - une plate-forme conventionnelle à trois pieds sur vérin

la tour - un treillis tubulaire triangulaire avec un minimum de membrures

la fondation - trois profonds caissons de pied ancrés à un niveau de fondation convenable au fond de la mer.

Ceci combine les meilleures caractéristiques de chaque élément et les avantages d'une plate-forme auto-élévatrice et à embase.

ZUSAMMENFASSUNG

Die neue hybride Konstruktion "TRICON" lässt sich in drei Hauptabschnitte unterteilen:

oben befindet sich die übliche Plattform, die an drei Beinen auf und ab bewegt werden kann

der Turm ist eine fachwerkartige Rohrrahmenkonstruktion mit einem Minimum an Stäben

die Gründung erfolgt über drei Tiefcaissons, welche in den Seegrund bis zur tragfähigen Bodenschicht abgesenkt werden.

Damit werden gleichzeitig die Vorteile einer Hubinsel und einer Schwergewichts-Plattform realisiert.

IVa

Synthesis and Conclusions

Synthèse et conclusions

Synthese und Schlussfolgerungen

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Offshore Structures

Mr. Chairman, Ladies and Gentlemen,

We have just heard some additional highly interesting discussions on the subject of offshore structures which further emphasized the complexity of operating in offshore regions.

We would make a serious mistake if we would consider this field as an exclusively structural engineering territory. There is no question that the structural engineering input is of vital importance in developing an ultimately satisfactory structure. However, offshore engineering design requires the input of the oceanographer, who defines the sea state; the ocean engineer, who translates the ocean environment into hydrodynamic loads; the naval architect, who considers the stability and platform motions of both floating drilling and production systems and offshore towers during the towing and upending phases; the geotechnical engineer, who provides the necessary data to develop the foundation design; and the fabricator or contractor, who ultimately creates the system.

Each of these areas represents a challenge to the expert:

Variability and Environmental Influence on Wave Conditions

Most design wave spectra are based on open sea observations or at least, observations taken in areas of relatively open water. Since sea conditions can be strongly affected by bottom and shore topography, some attention should be given to ascertaining how such factors may modify wave conditions in a specific design condition, and to sensitivity studies which are aimed at finding the effect on a structure of anticipated variations about the design wave conditions.

Interaction of Wind Velocity Profile and Waves Impinging on a Structure

If a structure is exposed to storm waves of height similar to the dimensions of the above-water portion of the structure, the wind velocity profile and wind force acting on the structure vary with time. This wave influence on the design wind characteristics will

be superimposed on the natural gustiness of the storm wind environment and undoubtedly have an influence on both the mean and time dependent wind forces.

Correlation and Proper Simultaneous Treatment of Waves and Current

Important questions here involve the appropriate way of representing the combined flow field and the computation on bodies exposed simultaneously to waves and currents.

Hydrodynamic Loadings

Hydrodynamic loads, acting on the emerged structural members, are a result of the fluid motion associated with waves and current, and of the body motion induced in response to these forces, as well as earthquake and other external excitation. The combined structure-fluid motion effects are referred to by the terms "fluid-structure interaction" or "relative motion effects". The prediction of these fluid forces requires the solution of the complete fluid equations of motion (Navier-Stokes equations) with complicated boundary conditions, including the free surface, moving rigid boundaries of the structure, and the fixed boundaries, e.g. the ocean bottom. Complete solutions of this problem have been obtained in only a few elementary cases, and often substantial simplifications to the problem have been made. In the case of bodies whose dimensions are of the same order as the wave lengths, viscous forces are greatly exceeded by inertia forces, and may therefore be neglected in determining both the flow kinematics and the forces on the body. For bodies of small cross-section compared to the wave length, the pressure, velocity and acceleration field associated with wave motion is assumed unaffected by the presence of the body. The viscous drag forces and inertia or pressure forces are computed, using these kinematic quantities combined with imperical drag and inertia coefficients; the so-called Morrison formula. However, there is a range of body sizes in which both diffraction and viscous effects are important. It is in this area that a thorough understanding of the relative importance of these effects is essential. Furthermore, members which pierce the mean water surface are subjected to some other forces. These are the time-varying buoyancy and impact or slamming forces. The former is determined by geometric consideration, but the latter is not well understood at present. The wave impact force on above water structural members is similar to the slam force on a ship pitching in heavy seas or the impact of a seaplane when landing. However, it is necessary to recognize the differences between the phenoma which apply on the one hand in the case of a moving body contacting a moving wave surface, and on the other hand, a moving wave impinging on an essentially fixed body.

Wind Loading

Wind action on offshore platforms can contribute significantly to design loads. To evaluate the magnitude of wind loads, it is necessary to first predict the nature of the wind produced at the site, in terms of maximum mean wind velocity and the fluctuation of the wind velocity about this maximum mean value, i.e. gust characteristics, and second to transform these velocities into drag forces. Thi transformation is difficult to achieve realistically due to the very irregular geometry of the platform and its equipment. Conceptually, the wind force transfer function can be characterized as depending on the altitude and air density, time velocity direction characteristics

shape and effective area, motion response characteristics of platform, projected area, and variation of the force with velocity.

Ice Loadings

Ice loading on offshore platforms in the Arctic seas can be extremely severe. Ice islands and icebergs, which are of land origin, can easily destroy a platform. Fortunately this danger is limited since these are rare; therefore, no consideration of their effects is given in the design of offshore platforms. On the other hand, sea ice is frequently present and must be considered in the design. This ice, which forms in sheets, can be pushed into ridges, rafts, and hummocks, producing unconsolidated masses which later consolidate upon refreezing. The depth of this ice is highly variable, causing similar variations in the magnitude of force exerted on offshore platforms. This maximum force is limited by failure conditions at the edge of the ice mass; therefore, its magnitude depends on (1) the contact area, as determined by the thickness of the ice sheet and the maximum width of indentation caused by the structure during impact, (2) the aspect ratio of ice thickness to width of structure, (3) the shape of the structure, (4) the strength of the ice in unconfined compression, (5) the elastic properties of the ice, and (6) the conditions of ice as unfrozen or frozen, to the face of the structure.

Soil Loadings

Soil loadings can result from lateral and vertical loadings on foundation elements and develop rapidly, as a result of soil deformations in which the instantaneous and/or accumulative relative deformations are large. Such deformations could be developed from bottom soil movements induced by wave action, earthquakes and other similar flow slides or combination of the same. In addition, settlements, both laterally and vertically can occur during long periods, and may affect the foundation stability over the long time.

Earthquake Loadings

The earthquake loading is similar to the other principle loads acting on offshore structures in that the intensity of loading is influenced strongly by basic characteristics of the structure. However, in the case of the earthquake loading, the input forces depend on the mass and stiffness of the structure rather than on the exterior form, and by adjusting these parameters, particularly the stiffness, the designer can exert a great influence over the seismic loads a structure must resist. This fact is particularly significant in regard to the non-linear behavior of an offshore structure, because damages which occur in the structural system generally lead to reductions of stiffness which allow relaxation of the seismic input intensity. Thus, to a significant extent, earthquake damages tend to be self-limiting processes and the primary concern of the designer is to insure that the vertical load capacity of the structure is maintained as the earthquake damages occur.

Structural Modeling

With the loads defined, a mathematical model of the structural system must be established to permit the three-dimensional analysis of the internal forces and stresses. In the event of high overloads, such as produced by high magnitude earthquakes, this model must properly reflect the inelastic deformations produced. Further, in the

interest of providing a suitable working environment, motion of the deck must be predicted realistically. Only through refined and advanced analytical procedures and modeling techniques is it possible to accurately predict the structural response.

Foundation Modeling

To realistically model fixed platforms for three-dimensional analysis purposes, it is necessary that the foundation be properly modeled so that the structure-foundation and direction effects will be included in the results. As in the case of structural modeling, the techniques and methodologies used for this purpose vary considerably in practice. Basically there are three basic forms of foundation modeling for spread footings or mat foundations, namely: (1) constant discrete springs and dashpots for the soil foundation with the spring constants determined from elastic-static half-space theory and the dashpot coefficients having assigned values to represent material damping in the soil, (2) discrete springs and dashpots for the soil foundation with the spring constants and dashpot coefficients being frequency dependent in accordance with the elastic or viscoelastic dynamic half-space theory, and (3) finite element representations of a body of soil at the base of the structure. Foundation modeling for fixed towers supported on piles is most difficult to accomplish realistically. The techniques and methodologies used for this purpose range widely in practice, the simplest form being that of assuming full fixity for the piles at some specified effective depth in calculating lateral stiffness and assuming a distribution of the vertical load transfer from the pile to the soil in calculating vertical and rotational stiffnesses. For firm foundation conditions, the forces applied to the pile cap will be rapidly transferred to the soil with increasing depth into the foundation. In this case, pile cap impedance functions can be approximated using the impedance functions generated for a rigid massless circular disc supported on an elastic or viscoelastic half-space. For softer foundation conditions, the forces applied to the pile cap are transferred to the soil at deeper depths; thus, a more realistic representation of the load transfer from piles to soil is needed. This problem is a topic of basic research at the present time which should yield suitable pile foundation models in the near future.

Analysis Procedures

Dynamic analyses of linear fixed towers can be carried out in either the time domain or the frequency domain. Most analyses of this type, carried out in the past, have used the time domain approach. Recently, however, the frequency domain approach has become comparative through the development of Fast Fourier Transform techniques. If frequency dependent parameters are included in the overall fluid-structure-foundation system (e.g. frequency dependent hydrodynamic drag and inertia coeffecients and/or frequency dependent foundation impedance functions), a time domain solution is not possible, but the frequency domain approach can be used without difficulty.

Performance Criteria

In addition to general loading methodologies and structural and foundation modeling techniques, other criteria related to the detailed structural response (e.g. fatigue, non-linear behavior, failure) are equally essential in the design process. Based on his experience

in fields other than offshore engineering, the structural engineer is particularly qualified to establish detailed prototype response criteria for both elements, connections and material properties. Furthermore, the overall design may not only be governed by the ultimate performance aspect, but also by the behavior of the structure during the different phases of construction, tow-out and installation. In the past, these latter phases have been considered as structurally less significant than the actual design phase. However, present offshore developments require structural systems for water depths of 800 to 1,000 feet or more. The remoteness of these regions may require tows covering several thousands of miles between assembly yard and installation site. Also, upending processes at the installation site necessitate a careful assessment of the dynamic response during this stage. It is now essential not only to consider the platform stability, but also the dynamic structural response during these tow-out and upending phases. This aspect becomes particularly significant if one considers the duration of these tows under potentially most adverse weather conditions and the often hostile prevailing sea state at the site. Damages during this phase, be it permanent or of fatigue nature, are factors which need full consideration in formulating the final design.

The previous comments emphasize that offshore structural design reflects the combined effort of many engineering disciplines. To achieve an optimum design for both stationary and floating structures in a mostly hostile, highly random and often insufficiently known environment, requires an integrated approach involving the most advanced technologies.

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Comments by the Author of the Introductory Report

Remarques de l'auteur du rapport introductif

Bemerkungen des Verfassers des Einführungsberichtes

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Foundation Structures for Tall Buildings

This presentation in Working Session IV concerns the foundation structures for tall buildings. In the writer's Introductory Report (Ohsaki 1976), he has pointed out a number of problems associated with design of foundations for tall buildings.

The first problem is the heavy weight of a tall building which requires large bearing capacity of supporting soil stratum. Bearing capacities of sand stratum, gravel stratum, or heavily over-consolidated clayey stratum are usually in the range of 150 to 400 tons/sq.meter, as shown in Fig.1, allowing the

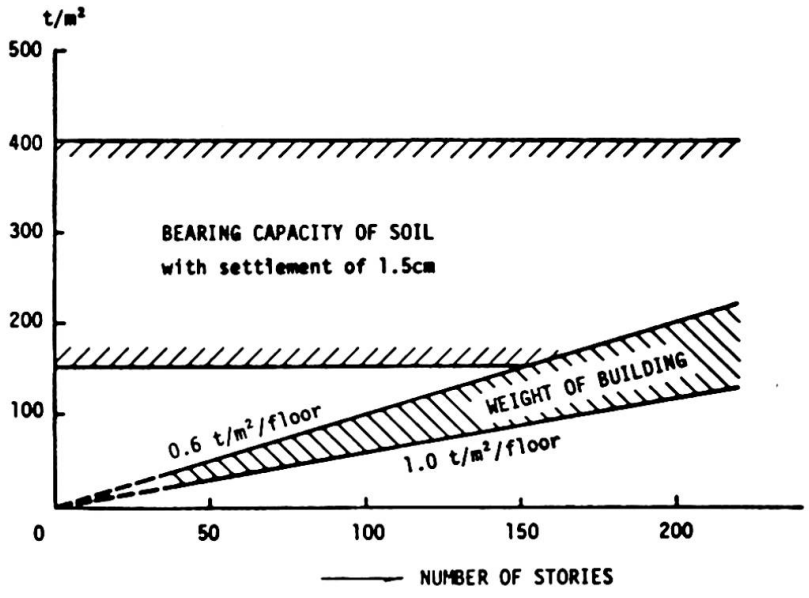

Fig.1

settlement of approximately 1.5 centimeters. On the other hand, unit weights of tall buildings of typical, steel construction are estimated to be in the range of 0.6 to 1.0 tons/sq.meter/floor. Therefore, the total weights of buildings are in the triangular range in Fig.1, which shows that there is no

serious problem in foundation design, if soil stratum of such strength can be encountered within the depth of a building basement.

If such stratum is located at a certain depth below the base of a building, a pile foundation can be used and, fortunately, no fundamental problem has been met so far in performance of pile foundation. The application of floating foundation may also be possible, which balances the weight of building and the weight of excavated soil so that no additional load would be applied to the supporting soil stratum. It is possible theoretically up to the building of approximately 40 stories; however, it seems that there has been no experience of applying the idea of floating foundation actually to high-rise buildings.

The second problem is the effects of wind force. As shown in Fig.2, when a building is subjected to wind force, the overturning moment causes an increase

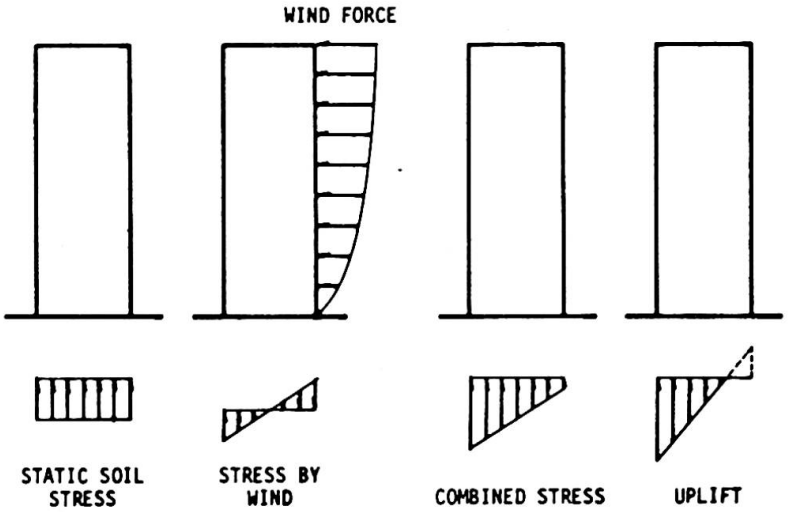

Fig.2

of soil reaction stress on one side and a decrease on the other side. Usually, as the combined stress is within twice the static stress, if Fig.1 is again considered, there would be no danger of failure of supporting soil. If overturning moment increases still more, uplift takes place as also shown in Fig.2, which indicates a tendency to make the structure unstable. However, the uplift can readily be prevented by providing setback as shown in Fig.3 and, when the setback can not be provided, anchoring shown in Fig.3 may be an effective way, but it has not been actually utilized for tall buildings up to the present time.

In a foundation slab, the outer part shaded in Fig.4 is most effective to resist the overturning moment, while the middle part is not so effective mechanically. Moreover, if permanent settlements are produced beneath the edge of a foundation slab under repeated wind loadings, the presence of the middle part is rather harmful causing the so-called riding effect, which will accelerate the rocking motion of the superstructure. In consideration of these and other facts, Schlaich and Otto suggest an idea of ring foundation for a tower structure, and describe in detail many examples of their design experiences of actual tower structures in their Preliminary Report (Schlaich & Otto 1976).

The last problem is the effects of seismic forces. In Japan of extremely high seismicity, seismic forces are usually larger than wind forces for typical buildings of less than approximately 50 stories. However, as a general shape of acceleration response spectrum in Fig.5 indicates, the earthquake input to tall and flexible buildings is rather small when compared with low and stiff

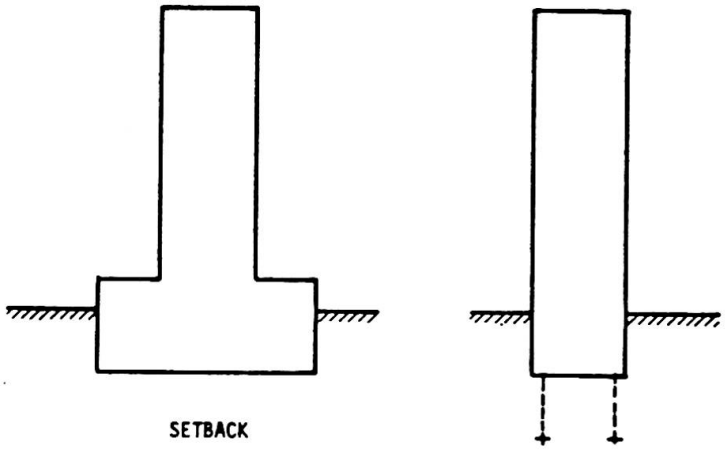

Fig.3

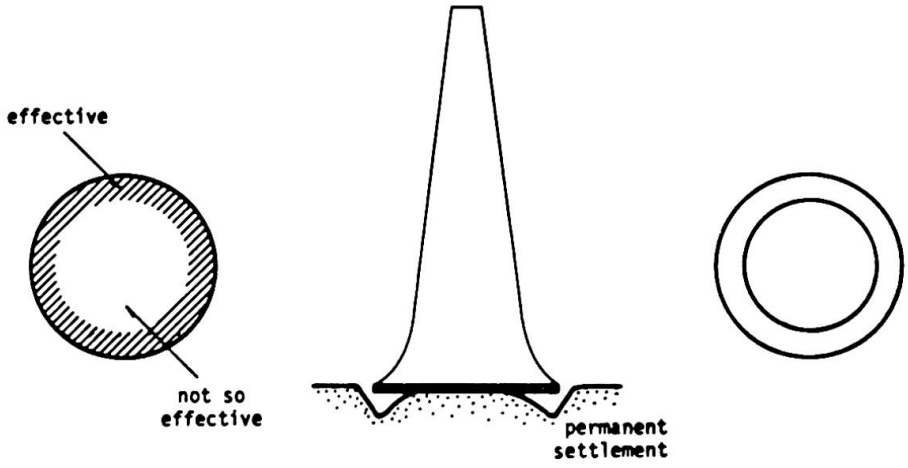

Fig.4

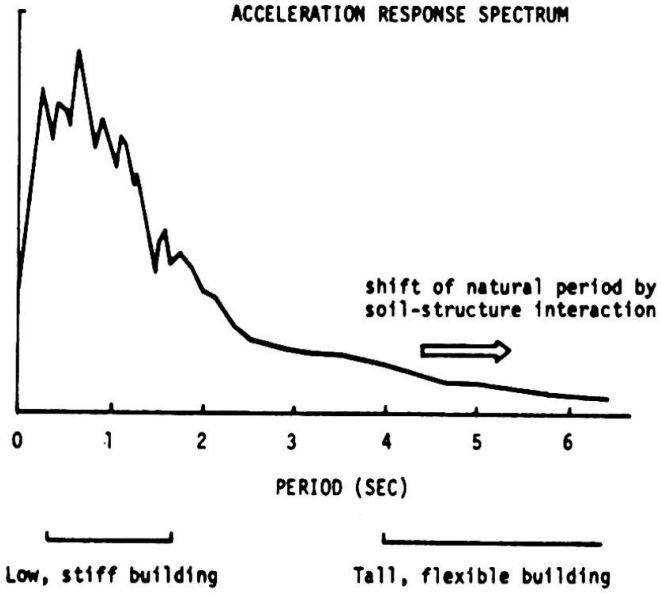

Fig.5

buildings. Moreover, the effect of soil-structure interaction tends to still reduce the input force, although the interaction effect is not important in general for flexible buildings.

In conclusion, there may be several problems in foundation design specific to tall buildings, they will not be of great difficulty to overcome unless soil conditions are extraordinarily unfavorable.

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Schlaich, J. & U. Otto (1976) : Zur Gründung hoher Stahlbetontürme, Preliminary Report, 10th Congress, Tokyo, International Association for Bridge and Structural Engineering, pp.349-355

Foundation Structure of the CN Tower (Toronto)

Fondation de la tour de télécommunication "CN Tower" (Toronto)

Fundation des Fernmeldeturms "CN Tower" (Toronto)

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1. Description of Structure

The recently completed CN Tower in Toronto (Canada) is the highest free standing structure of the world rising to a height of 550 m (Fig. 1). It consists of a tapered, Y-shaped, multi-cellular concrete shaft up to 447 m and a steel antenna mast of 103 m covered by fiber reinforced plastic cylinders (Fig. 2).

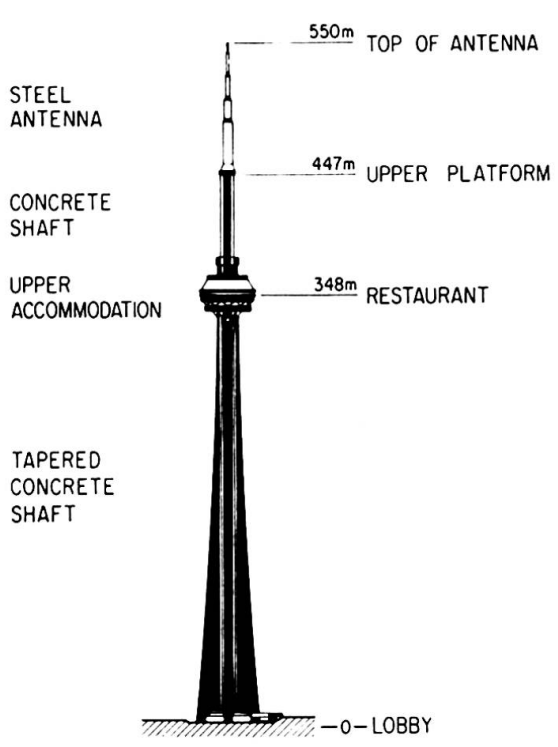


Fig.1: CN Tower, Toronto

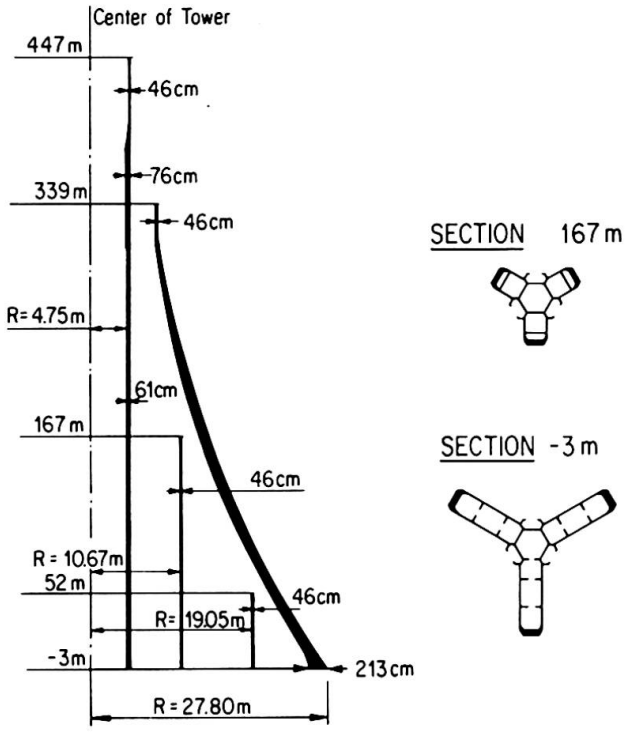


Fig.2: Typical Dimensions of Tower Shaft

The tower houses various telecommunication transmission facilities. In addition a restaurant and observation decks at the 348 m level and an upper observation platform at the 447 m level are open to the general public.

The selection of the shape and the cross section of the concrete shaft was influenced not only by statical and architectural considerations. Due to the fact that only three walls, i.e. the outer walls of the three legs are sloping the application of slip forms in casting the entire shaft became possible. Vertical prestressing of the tower resulted in a number of advantages and cost saving:

- No tensile stresses under the 50 year wind.
- About 75% less steel weight for the same strength, cost savings relative to the unit prices of prestressing and ordinary reinforcement.
- Concentrated forces, no splices, less congestions due to reinforcements.
- Great reduction in the quantity of ordinary steel to be placed during the slipping operation of the formwork.

2. Foundation Base

The foundation base consists of the Meaford-Dundas shale formation which is typical for the downtown area of Toronto. Some twenty borings and four observation test wells of 76 cm diameter were made, one reaching a depth of 40 m. Under an unconsolidated fill of about 9 m the bedrock comprises a sedimentary sequence of horizontally thinly bedded or laminated shales, calcareous shales and limestones. Some narrow zones of rubbled shale, 15 to 45 cm thick occur. Some clay seams up to 1.5 cm thick are also present. These seams together with the presence of vertical cracks in the shales lead to the use of prestressing of the foundation structure as will be explained later on. The ground water level is at a depth of about 6 m.

In the following Table 1 the unconfined compression strength of the rock samples is listed.

	Shale	Calcareous Shale	Limestone
Amount of Sequence			
Unconfined Compression Strength	84 kg/cm^2	190 kg/cm^2	1100 to 2000 kg/cm^2

Table 1: Compression Strength of Rock Base

3. Foundation Structure

The foundation structure is shown in Fig. 3. It follows the Y-shape of the tower shaft. The hollow core slab has a thickness of 5.50 m. The live anchors of the prestressing cables of the tower walls are located in these caverns. The prestressing operations for these cables, up to 450 m long, could hence be executed in this protected area without interference of the work on the platform of the slip form.

The outer walls of the three tower legs are inclined with respect to the vertical as illustrated in Fig. 4. The horizontal components of the forces in these walls due to dead load and wind moment produce considerable tensile forces in the foundation slab. Resisting forces can be built up by (1) the reinforcement of the

slab, (2) friction forces between slab and rock and (3) resistance of the rock wall around the perimeter.

It was decided to take up these permantly acting spreading forces by posttensioned cables, Fig. 5. Frictional forces (2) could not be relied upon because of the above mentioned clay seams and high pore water pressures in response to dynamic wind forces. Resisting forces (3) along the perimeter would only be activated after substantial horizontal movements. Hence, in order to prevent vertical cracking of the foundation slab and the tower walls above as well as of the rock layers below prestressing was chosen.

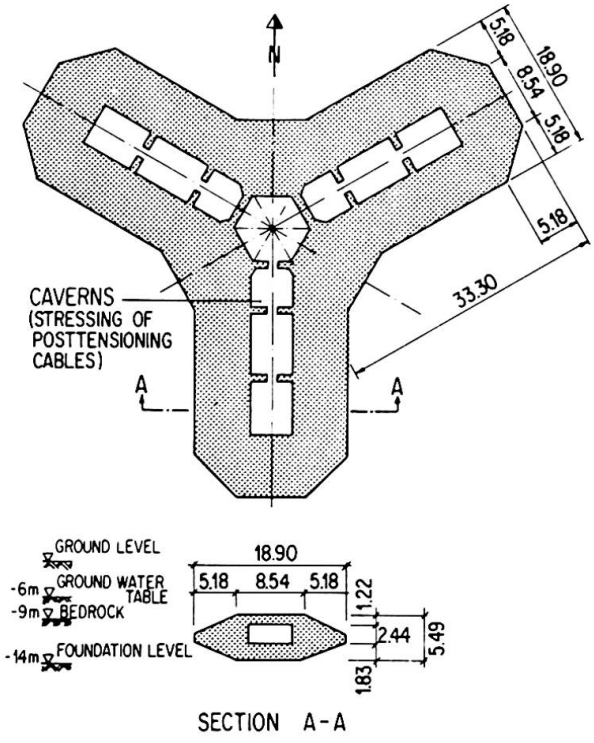


Fig.3: Foundation Slab

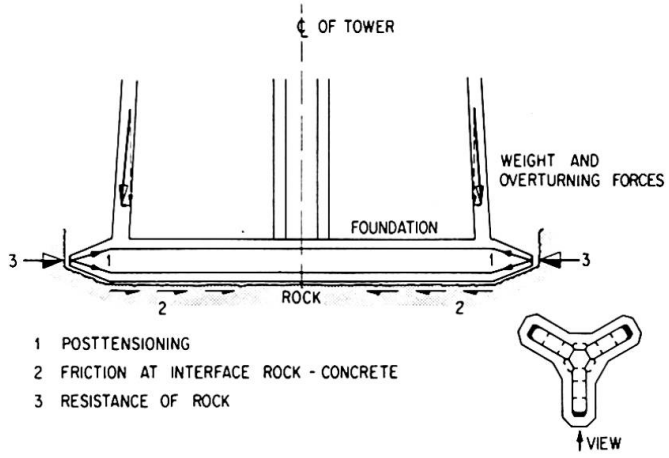


Fig.4: Horizontal Forces on Foundation

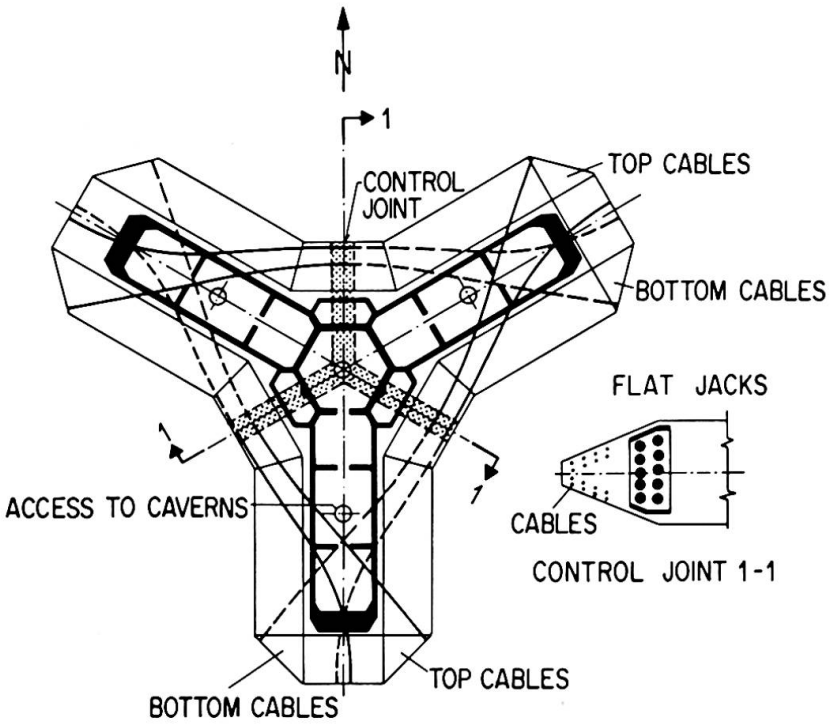


Fig.5: Prestressing of Foundation Slab

Bands of prestressing cables in the arrangement of Fig. 5 were placed running below and above the slab caverns. Control joints 1-1 separated the three legs completely. A sequence for the posttensioning operation was developed such that the outer edges of the three legs remained fixed. In this way vertical cracks in the shale at these critical points and a possible deterioration of its resistance could be avoided. By activating the active anchors at both cable ends and the flat jacks in the control joints in an appropriate sequence this condition could be met. A subsequent analysis of the jack forces showed that the rock base provided practically no frictional resistance as anticipated.

In order to reduce volume changes due to hydration heat and shrinkage a modest dosage of cement of 210 kg per 1 m^3 of concrete was used.

Tapering edges (see cross section Fig. 3) around the foundation slab were selected on the basis of a stress analysis on the interaction between slab and rock. In such a way a considerable reduction of the local shear stresses along the edges was achieved and the possibility of vertical cracking of the shale greatly reduced.

4. Summary of Design Data

The most pertinent results of the analysis of the foundation structure are as follows:

Loading

Wind: Predicted extreme mean hourly gradient wind speed, 50 years return period:

$v_{50} = 167$ km/h

Base Forces, 50 years period:

Moment $M_{50} = 4.42 \cdot 10^5$ m•to

Shear $V_{50} = 1800$ to

During the initial design period the exact dimensions and contours of the antenna mast, the upper and lower accommodation levels and other details were not yet fixed. Hence, 25% higher reference values were selected for making the design calculations. This corresponds also to about a 10% increase in the wind speed.

Reference Forces:

$M_{ref} = 1.25\ M_{50} = 5.52 \cdot 10^5$ m•to

$V_{ref} = 1.25\ V_{50} = 2250$ to

Foundation Stresses

1. Dead Load + Live Loads + Buoyancy

   Normal Force $N = 94'800$ to

   Average Stress $\sigma = 5.5$ kg/cm^2

2. 0.9 (Dead Load + Live Loads + Buoyancy)
   Reference Base Moment M_{ref}

   $N = 85'300$ to

   $M_{ref} = 5.52 \cdot 10^5$ m•to

   Max. Edge Stress $\sigma = 10.5$ kg/cm^2

Overturning Moment

In Fig. 6 the conditions are listed. A conservative estimate for the maximum edge stress σ_{max} = 84 kg/cm² equal to the unconfined compression strength of the shale (Table 1) is used. Assuming a normal force due to 0.9 (Dead Load + Live Loads + Buoyancy), N = 85'300 to, the following value results:

Overturning Moment:

$$M_{over} = 1.68 \cdot 10^6 \text{ m·to}$$

Hence, the safety margin becomes:

$$s = \frac{M_{over}}{M_{ref}} \cong 3$$

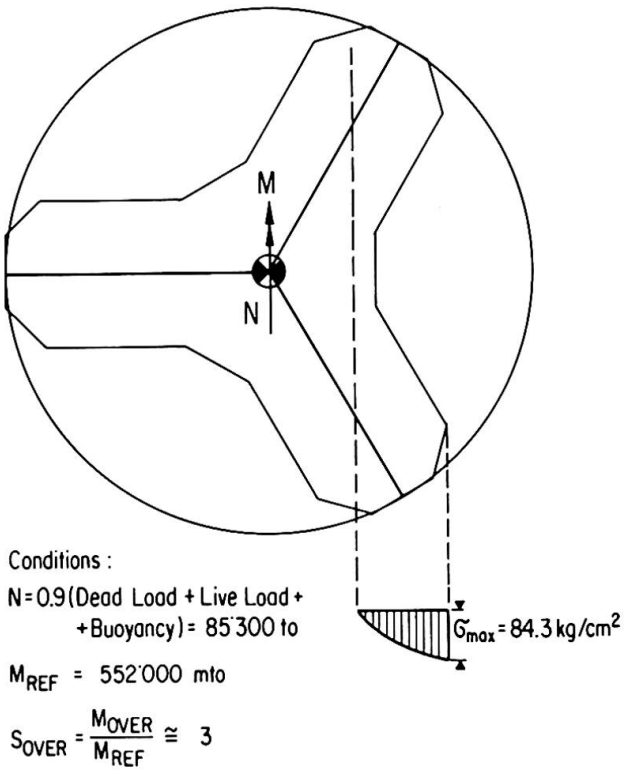


Fig.6: Overturning Moment

5. Acknowledgement

CN Tower Ltd., Toronto, are owners of the structure. The architects were John Andrews and Webb, Zerafa, Menkès, Housden, Toronto. For the structural design and the technical supervision R.R. Nicolet and Associates, Montreal, with F. Knoll in direct charge, were responsible. The Boundary Layer Wind-Tunnel Laboratory, University of Western Ontario, London, Ontario, was charged with the analysis of wind loads. The writer was consultant to CN Tower Ltd. and R.R. Nicolet and Associates and reviewed the structural design. He expresses his sincere appreciation to all parties involved for a most effective and stimulating collaboration.

6. Reference

Franz Knoll: "Structural Design Concepts for the Canadian National Tower, Toronto, Canada". Canadian Journal of Civil Engineering, Vol. 2, No. 2, 1975, pp. 123 to 137.

SUMMARY

The 550 m high CN Tower (447 m concrete shaft, 103 m steel antenna mast) is presently the highest free standing structure of the world. The Y-shaped foundation slab resting on a shale formation is prestressed. The reason for prestressing, the arrangement and stressing of the cables and some pertinent design data are presented.

RESUME

La "CN Tower", de 550 m de hauteur (la colonne en béton armé mesure 447 m et l'antenne métallique 103 m) est aujourd'hui l'ouvrage le plus haut du monde. La dalle de fondation précontrainte repose sur une couche de schistes argileux. Les raisons en faveur de la précontrainte, la disposition et la tension des câbles ainsi que différents aspects importants du dimensionnement sont présentés.

ZUSAMMENFASSUNG

Der 55o m hohe CN Tower (447 m Beton-Schaft, 103 m Antenne aus Stahl) ist heute das höchste freistehende Bauwerk der Welt. Die Fundationsplatte in Y-Form ruht auf einer Formation von Schiefertone und ist vorgespannt. Die Gründe für die Vorspannung, die Auslegung und das Spannen der Kabel sowie weitere wichtige Angaben über die Bemessung, werden aufgeführt.

Der Richtfunkturm des Fernmeldezentrums Arsenal in Wien

The Radio Link Tower of the Telecommunications Centre Arsenal in Vienna

La tour radiophare du centre des télécommunications "Arsenal", à Vienne

ROBERT KRAPFENBAUER
o. Prof. Dipl.-Ing. Dr. techn.
Ingenieur Konsulent
Wien, Österreich

Dieses bedeutende Turmbauwerk wurde im Auftrage der Post- und Telegraphendirektion Wien im Rahmen eines Fernmeldezentrums als größtes Postbauvorhaben Österreichs errichtet. Der entwerfende Architekt war Dr. Kurt Eckel, der Statiker Prof. Dr. Robert Krapfenbauer.

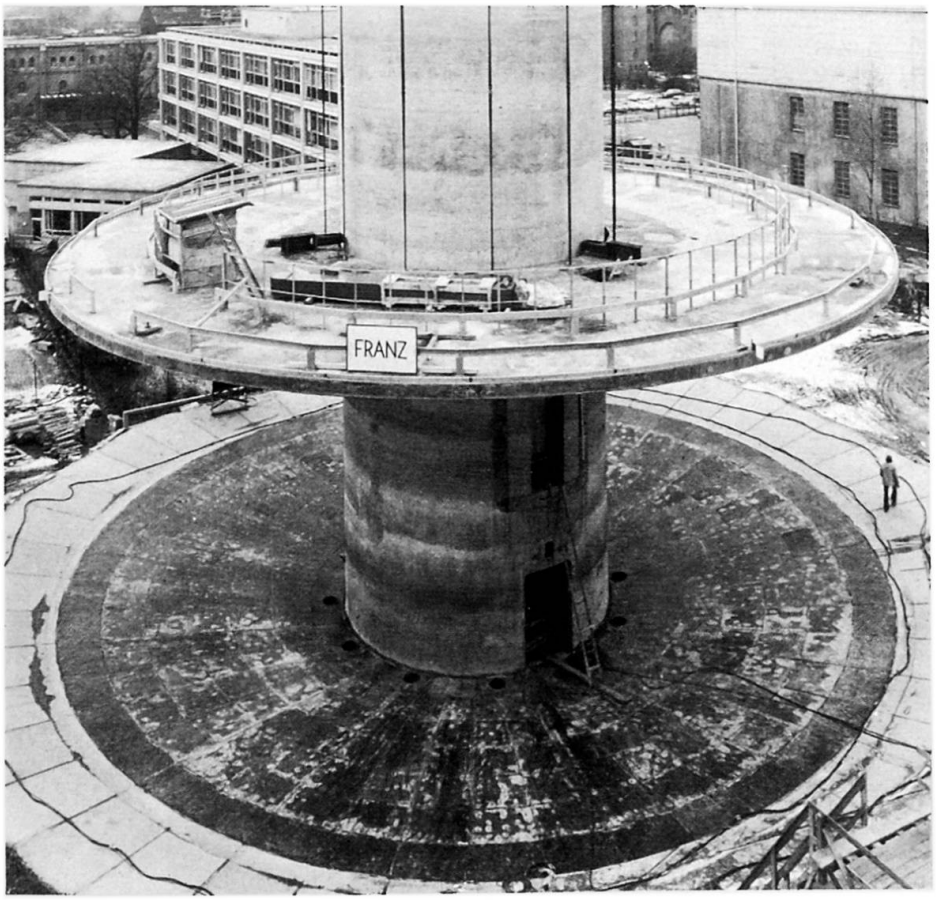
Die gesamte Anlage umfaßt zwei Betriebsgebäude von 12 bzw. 7 Geschossen, verbunden durch einen Verkehrskern, einen Bürotrakt, einen Zwischentrakt und den Richtfunkturm. Die verbaute Fläche beträgt 23.000 m^2. Der gesamte umbaute Raum beträgt 289.000 m^3. Das Turmbauwerk aus Stahlbeton hat eine Höhe von 136,6 m mit einer aufgesetzten 18,4 m hohen Stahlantenne, so daß die Gesamthöhe 155 m beträgt.

Der Richtfunkturm des Fernmeldezentralgebäudes Arsenal ist vom Postbetrieb her als hochgradig setzungsempfindliches Bauwerk aufzufassen. Für die Konstruktion des Gesamtbauwerkes war zur sicheren Gewährleistung des Fernmeldebetriebes die Forderung nach einem maximalen Endtangentenwinkel von 0,3 Altgrad ausschlaggebend. Ein nicht unerheblicher Anteil der Endtangentenneigung stammt von der Schrägstellung des Fundamentes unter der Wirkung des Einspannmomentes infolge Windwirkung auf den Turm.

Der Baugrund besteht aus einer im Mittel 4,5 m starken Anschüttung, der 8,5 m starken Arsenalschotterdecke und darunter liegenden tonigen Mittelpannon-Schichten. Aufgrund der Bodenanalyse und der Setzungsberechnungen konnte mit einer mittleren zulässigen Bodenpressung von $\sigma_m = 5,5$ kg/cm^2 gerechnet werden. Die Endsetzungen wurden mit 4 cm ermittelt.

Geophysikalische Untersuchungen ergaben relativ horizontal verlaufende Pannon-Schichten über eine Bruchstaffel des Hauptbruches des Wiener Beckens. Das Vorhandensein dieser Bruchstaffel weist auf die besonders bebengefährdete Lage des Baugrundes hin, in der Standberechnung mußte mit einer Horizontalbeschleunigung von g/40, dem fünffachen Mindestwert der österreichischen Normen, gerechnet werden.

Die Gesamtlast von 13.120 Mp ergibt bei einer Aufstandsfläche von 328 m^2 eine mittlere Bodenpressung von 4,0 kp/cm^2. Das maximal auftretende Moment aus der Überlagerung des Windmomentes mit den Momenten aus einseitiger Nutzlast, Bauimperfektionen und Momenten II. Ordnung beträgt 28.500 Mpm und bringt mit der maximalen Vertikallast eine Randpressung von 7,0 kg/cm^2. Selbst bei reinem Eigengewicht, ohne Erdauflast auf dem Fundament und maximalem Windangriff entsteht kein Klaffen in der Sohlfuge.



ANHEBEN DER PLATTFORMEN BEIM RICHTFUNKTURM ARSENAL

Der Gründungskörper des Fernmeldeturmes Arsenal besteht aus einer im Grundriß kreisförmigen Bohrpfahlwand und dem eigentlichen Turmfundament aus Stahlbeton B 300 im Inneren der kreiszylindrischen Bohrpfahlwand. Die Fundamentsohle liegt 8 m und die Pfahlspitzen 16 m unter Gelände.

Die Bohrpfahlwand mit einem Innendurchmesser von 21,30 Meter wird aus 90 cm starken, 16 m langen, überschnittenen Bohrpfählen gebildet. Sie diente einerseits als Baugrubenumschließung für die Erstellung des eigentlichen Fundamentkörpers, andererseits wird mit ihr eine 2 Meter starke Störzone in den oberflächennahen Bodenschichten durchfahren, so daß eine Lastübertragung des Turmbauwerkes auf eine 3 Meter starke, annähernd horizontal liegende Schicht des Wiener Tegels erfolgt.

DER RICHTFUNKTURM DES FERNMELDEZENTRUMS WIEN-ARSENAL

Das Fundament selbst wird aus einer 1,20 Meter starken kreisförmigen Sohlplatte, einer 70 cm starken Zylinderschale in Verlängerung des Turmschaftes, einer 65 cm starken, außenliegenden Kegelschale mit einer Neigung der Erzeugenden zu der Horizontalen von 52,15° und einer obenliegenden 80 cm starken horizontalen Abschlußscheibe gebildet.

Im Inneren des Fundamentkörpers befindet sich die Putzgrube für die Aufzugsanlage und ein Lagerplatz für Ausgleichsgewichte zur Kompensation einseitiger Nutzlasten auf den Turmplattformen. Der Hohlraum zwischen Kegel- und Zylinderschale ist überdies zu Inspektionszwecken begehbar.

Während des Bauvorganges des Turmbauwerkes und nach dessen Fertigstellung wurden laufend Messungen vorgenommen. Als Gesamtsetzung des Turmbauwerkes ergaben sich nach zwei Jahren Bestand 1,5 cm. Schwingungsmessungen ergaben genau den rechnerisch ermittelten Wert der Grundfrequenz, Messungen der Bewegungen des Turmes bei Windgeschwindigkeiten von 110 km/h ergaben Werte weit unter den vom Postbetrieb festgesetzten Grenzwerten. Es konnte somit nachträglich die Wahl der Fundierung des Fernmeldeturmes in Wien-Arsenal in technischer und wirtschaftlicher Hinsicht voll bestätigt werden.

Der Turmschaft wurde sodann auf dem Fundamentkörper in Gleitbauweise errichtet. Der Schaft hat einen konstanten Durchmesser von 8,4 m; die Wanddicke wird vom Erdniveau bis zur Spitze in drei Stufen reduziert (von 70 cm über 50 cm bis auf 35 cm). Auf dem Stahlbetonschaft liegt eine Betonplatte von einer Dicke 1,50 bis 1,85 m auf, die den Stahlmast trägt.

Auf dem Turm sind vier scheibenförmige Antennenplattformen sowie ein Betriebsgeschoß in ihrer Mitte angebracht. Zwei der Stahlbetonplattformen haben Durchmesser von je 25,4 m und wiegen je 470 Tonnen, zwei haben Durchmesser von 30,20 m und wiegen je 600 Tonnen. Alle vier Plattformen haben vorgefertigte Versteifungsrippen und einen vorgespannten Umfassungsring an der Außenkante. Der untere Teil der Plattformen besteht aus einer flachen, konischen Schale, die zusammen mit den vorgefertigten inneren Radialrippen und der oberen Deckplatte ein steifes Raumtragwerk bildet. Die Dicke der konischen Schalen liegt zwischen 8 und 9 cm. Die Anordnung der Schalen auf dem Stahlbetonschaft ist

(von unten nach oben) folgende: eine kleinere, eine größere Plattform, das Betriebsgeschoß, eine größere und schließlich oben wieder eine kleinere Plattform.

Das Betriebsgeschoß, genannt Gondel, ist eine Stahlkonstruktion mit Außenverkleidung aus Aluminium-Blech und Glas. Der Durchmesser dieses Geschosses beträgt 35 m.

Sowohl die vier Antennenplattformen als das Betriebsgeschoß wurden auf Erdniveau gegossen und montiert und schließlich mittels der Lift-Slab-Methode auf ihre endgültige Höhe gebracht. Dies war erforderlich infolge der hohen Windgeschwindigkeiten von 75 bis 90 mph an der Turmspitze (100 m), die für eine Herstellung an Ort zu gefährlich gewesen wären.

Der Bauvorgang war folgender:
Für die beiden oberen Plattformen wurden Schalungen angefertigt, die Plattformen hieraufgegossen und gehoben. Als nächstes wurde das Betriebsgeschoß auf provisorischen Stützen in geringer Höhe über dem Erdboden montiert und geliftet. Die Schalungen für die verbleibenden beiden unteren Stahlbetonscheiben blieben zur Wiederverwendung erhalten. Nach dem Hochziehen wurde jede Plattform mittels zwölf stählerner Halterungen am Turm befestigt. Durch die Anwendung der Lift-Slab-Methode konnte die Konstruktionszeit auf drei Jahre beschränkt werden.

ZUSAMMENFASSUNG

Der Richtfunkturm des Fernmeldezentrums Arsenal in Wien stellt ein Turmbauwerk von 136,6 m Höhe mit einer aufgesetzten 18,4 m hohen Stahlantenne dar, sodass die Gesamthöhe 155 m beträgt. Der Gründungskörper für den Turm besteht aus einer im Grundriss kreisförmigen Bohrpfahlwand und dem eigentlichen Turmfundament aus Stahlbeton im Innern. Die Fundamentsohle liegt 8 m und die Pfahlspitzen 16 m unter Gelände. Der Turmschaft aus Stahlbeton wurde in Gleitbauweise errichtet; er trägt vier Antennenplattformen aus Stahlbeton und ein Betriebsgeschoss in Stahlkonstruktion. Diese fünf Teile wurden auf Erdniveau hergestellt und im Lift-Slab-Verfahren hochgezogen.

SUMMARY

The Radio Link Tower of the Telecommunications Centre "Arsenal" in Vienna is a tower structure of 136,6 m height with a 18,4 m high steel antenna set on it, so that the total height is 155 m. The foundation body for the tower consists of a circular wall of boring piles and the foundation itself of reinforced concrete within this wall. The sole of the foundation is 8 m and the pile ends

lie 16 m below ground-level. The tower shaft of reinforced concrete was built by the sliding consctruction method; it carries four antenna platforms of reinforced concrete and one equipment pod in steel construction. These five parts were made and assembled on ground level and lifted by means of the lift-slab-method.

RESUME

La tour radiophare du centre des télécommunications "Arsenal" à Vienne est une tour de 136,6 m de hauteur avec une antenne métallique de 18,4 m à son sommet de sorte que la hauteur totale est de 155 m. La fondation de la tour consiste en une paroi circulaire formée de pieux, et le corps propre de la fondation à l'intérieur de ce cercle est en béton armé. La base de la fondation est à une profondeur de 8 m, les pointes des pieux à 16 m de profondeur. La tour est en béton armé; elle porte 4 plate-formes en béton armé pour les antennes et un étage de service en construction métallique. Les 5 parties ont été construites au niveau du sol et levées par la méthode "Lift-Slab".

Comments by the Author of the Introductory Report

Remarques de l'auteur du rapport introductif

Bemerkungen des Verfassers des Einführungsberichtes

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Foundation Structures for Long Span Bridges

1st Paper

The foundation of the Seine River Bridge Brotonne, a cable-stayed bridge of prestressed concrete with a main span of 320 m (fig. 1) (reported by J. L. Brault and J. Mathivat) is in different ways interesting. The bridge deck is about 54 m above water-level, the single pylon in the central axis of the bridge is 70 m high; pylon and pier together have a total height of 125 m (fig. 2). At the foot of the pier there act vertical loads of 17 300 tons and a transverse bending-moment due to wind forces of 31 800 tm. The French engineers chose a single circular column with a diameter of only 12.5 m and a depth of 35 m for the foundation of the tower-like piers of this very great bridge (fig. 3). The upper 9 m of the soil are soft, underlying are 8 m of alluvial sandy clay followed by 10 m of sand and gravel, resting on chalk with a compressive strength between 20 and 80 kp/cm^2 . The column of the foundation goes 8 m deep into this chalk corresponding to only 2/3 of her diameter.

For carrying the load and moment, the horizontal earth resistance was taken into account in spite of the fact that the stiffness coefficient of the 8 m thick alluvial soil is only 0.12 kp/cm^3 and that of the sand-gravel only 1.5 kp/cm^3. The chalk also has a rather small stiffness coefficient for horizontal pressure of 8 kp/cm^3. With these assumptions, the soil pressure distribution was calculated as shown in fig. 4 with a maximum horizontal pressure above the chalk of 2.6 and 2.8 kp/cm^2 . In the base section of the column, there is a remaining bending moment of 18 000 tm, which results in large differences of vertical soil pressure with a maximum value at the edge of 23.8 kp/cm^2. At this level, the stiffness coefficient of the chalk for vertical pressure would, therefore, cause a considerable deformation.

This is a daring foundation, however, one can expect that it will behave well, because the extreme wind loads caused by extraordinary gales in costal environment are relatively rare and their peak gusts, which develop highest wind velocities, last usually only one or two seconds. One knows that the statically calculated soil pressure will not realize by such short-time load attacks due to the mass inertia of the big structure and of the soil. Therefore the base of the foundation will mainly have to carry the vertical load and will not get to feel much of the wind loads.

But not only the small dimensions of the foundation of this large bridge are interesting but also the construction method. The construction was started with a self-supporting slurry trench wall acting as a cofferdam , built with 80 cm thick unreinforced concrete to a depth of 31 m (fig. 5) intruding 4 m deep into the chalk. The material inside this cylinder-cofferdam was dredged into the chalk and the concrete for the base could be executed in the dry. In this way it was also simple to construct the thick reinforced concrete walls of the main column.

This example demonstrates how economical such foundations of large bridges can be built, if one makes good use not only of the vertical bearing capacity of the soil, but also of the horizontal resistance. Hereto it is necessary to investigate reliably the data for soil behaviour, as it has been done for this bridge by several core drillings with 120 mm diameter to a depth of 50 m.

2nd Paper

In the second report, the Japanese engineers M. Ohashi, S. Kashima and O. Yoshida describe the foundation of the towers of the suspension bridge with a main span of 870 m, the Ohnaruto Bridge, which belongs to the system of bridges which shall connect the main island with Shikoku Island. The towers stand in shallow water of only 3 m depth on sandstone, the construction of the foundation will, however, be influenced by strong tidal currents with velocities up to 5 m/sec.

At the beginning it is reported that the Japanese turned during the last 12 years more and more to pile foundations for their large bridges. At the Ohnaruto Bridge, there are 9 big piles in steel pipes with diameters of 4.6 and 7.2 m under each tower leg (fig. 6), which intrude 15 m deep into the sandstone rock. The solid concrete pile cap has a thickness of 9 m. The caps for the two groups of piles are connected with a 6 m thick slab of reinforced concrete.

The multi-column-foundation was analysed as a space frame, the pile legs being spring-supported horizontally and vertically to imitate the response of the rock. The analysis was checked by model tests and by large scale models on the real ground.

With regard to earthquakes, dynamic model tests were also conducted not only with the pile-foundation but for comparison also with solid block-foundation.

The result was, that the multi-column-foundation shows larger deformations but also higher natural frequencies than the solid block-foundation (fig. 7). The natural frequency of the foundation is, on the other side, so different from the frequency of the steel towers, that the dynamic analysis of the tower and of the foundation can be done individually. The first mode of vibration is dominant, because the pile cap is so heavy.

The analysis as space frame gave a good agreement with model measurements. It was found also that the horizontal spring constant of the ground related to the total multi-pile-foundation is equal to that of one pile multiplied by the number of piles.

3rd Paper

Pile-foundation for long-spanned bridges are also described by the Russian engineers K.S. Silin, N.M. Glotov, V.N. Kutzenko and G.P. Solovyev (fig. 8). They often use piles made of prefabricated reinforced concrete tubes with a diameter of 3 m and a wall thickness of only 12 cm. 6 m long elements are flanged together with bolts. These large pipes are driven into the alluvial ground by vibration and simultaneous dredging. If the foundation reaches rock, then 3 to 4 m deep holes are drilled into the bed-rock by the use of a special turbo-drilling machine, which was developed for the oil industry of the USSR. After cleaning, the hole and the concrete pipe are filled with heavily reinforced concrete.

For a large railroad bridge, inclined drilled piles with steel pipes of 1.4 m diameter had been used, which reach to 40 m depth under the water level and end in dense clay (fig. 9). At the foot of the pile a conical cave is cut out of the clay with a diameter up to 3.5 m in order to increase the bearing area of the pile. This hole was filled with bentonite slurry to protect it against possible soil slides. After introducing the reinforcing cage, the concrete was filled in through pipes. The allowable capacity of such a pile is 800 tons, test loadings went up to two times this value.

4th Paper

In the fourth report, the Japanese engineers T. Okubo, K. Komada, K. Yahagi and M. Okahara describe a new type of foundation in deep water with sheet piling, using steel pipes with diameters up to 1.2 m and wall thicknesses up to 19 mm (fig. 10).

At the outer steel pipe faces, there are two small diameter steel pipes with a slot, which are used to guide each following pile during the driving process. The voids of the small pipes are filled with mortar in order to get a stiff and tight connection between the pipes.

The flexural stiffness of the pipes allows to use the structure as a cofferdam. With such pipes, ring shaped or oval or rectangular, foundation arrangements can be made in sizes suitable for bridge piers (fig. 11).

The authors describe different possibilities of application (fig. 12). The footing of a bridge pier can be placed directly on top of the steel pipes above the water level. The water can also be pumped out of the interior of the sheet piling to a certain depth and steel brackets can be welded to the pipes for carrying the footing of the pier. After concreting the pier, the sheet piling above this level can be cut off.

The authors describe how they calculate such sheet pile foundations by means of a finite strip method, using different coefficients of the sliding resistance in the connection between adjacent pipes and assuming different distribution of the soil pressure. The results of these analyses have been compared with measurements at a large scale foundation with circular pile arrangement. The analytical result is strongly influenced by the amount of sliding resistance

between the steel pipes and the comparison with measurements is not satisfactory. The authors conclude that more rational design methods should be developed and good design standards should be established.

This pipe-sheet pile foundation has already been used for 12 bridges with good success und with pile lengths up to 57 m. This special foundation can undoubtedly be further developed, a stiff connection between the pipes will be desirable in order to get a good use of the possible spatial resistance of such structures.

5th Paper

In the last contribution, the Japanese engineers S. Suzuki, M. Ishimaru, F. Nemeto and Y. Nojiri give a report on model tests for the foundation of the main pier of the Hamana Ohhashi Bridge, which has with 240 m the longest span of prestressed concrete box girder bridges. The foundation has to carry 27 000 tons vertical load and must be able to resist the horizontal inertial forces caused by the heavy superstructure during earthquakes, which result in bending moments longitudinally of 181 000 tm and transversely of 188 000 tm.

A rectangular multi-cell box caisson with slide lengths of 26 / 19 m and a depth of 30 m was adopted (fig. 13). The piers on top of this caisson are 20 m high.

Two models of acrylic resin were made. At the first model surface strains were measured in the usual way. The second model was used for three-dimensional photoelastic stress analysis, it was heated up to 130° C, loaded and cooled down. Thin slices of the model are then cut out which show the interior stress distribution in polarized light. The results (fig. 14) confirm the nonlinearity of stress diagrams in thick members. The distribution of shear stresses is not reported. Also the lengths of introduction of load attacks in prismatic bodies are confirmed.

The caisson was also analysed three-dimensionally with a finite element program which, however, was developed for thin plates, therefore this analysis is questionable. It was also used to study the influence of Poisson's ratio which is 0.39 for acrylic resin and 0.17 for concrete. The smaller value of the concrete leads to smaller flexural stresses in the thick upper plate of the caisson.

At this investigation, data are missing about the distribution of soil pressures, which must play a considerable part in the resistance of earthquake forces. The results show, on the other side, that the forces due to earthquakes do not cause serious stresses in such caisson foundations, even not under such heavy loads.

Remarks

From the contributions to the preliminary report we can gather that big progress is made with pile foundations. The diameters of the piles and their lengths are more and more increasing. The reports deal mainly with stress analysis, mathematically or by model tests, but say nothing about the

distribution of the load carrying reactions of the soil either by the pile foot or by friction along the pile's surfaces. One misses also considerations or means to secure the durability and life of the piles, for instance in salt water or in polluted harbour waters. One is further missing structural details, mainly details of the reinforcement of piles and pile caps. In my introductory report I have pointed to the splitting forces at thick reinforcing bars caused by high bond stresses as they develop in pile caps under large bridge loadings. Such splitting forces can be a considerable danger for the safety of such structures which most engineers do not yet realize.

The pile caps are usually large size solid concrete members in which the concrete stresses due to loads may be smaller than the stresses due to interior restraint caused by temperature or shrinkage differentials. For such thick concrete members prestressing to a degree to counteract tensile stresses under permanent load can lead to better behaviour and more safety than a heavy reinforcement with thick bars.

It is desirable that researchers and practising engineers being responsible for the safety of the structures, will pick up these problems and do not simply place several layers of thick reinforcing bars with small spacing into such pile caps. It can safely be predicted, that such reinforcements will not give the carrying capacity as calculated, but the concrete will split in the plane of the reinforcing bars before the structure reaches the required ultimate load.

Closing, I wish to draw your attention again to my introductory report for this Congress in which I have described a proposal for the foundation of the towers of a cable-stayed bridge with a span of 1 470 m for the crossing of the Straits of Messina which would have to be built in 95 m deep water. I proposed an economical ringshaped foundation of which the lower part would have to be built in a dry dock and then floated to the bridge site. The proposal was made in 1972 before Ekofisk in the North Sea was built. The extraordinary achievements of the engineers who have meanwhile built the oil tanks and drilling-platforms in the North Sea did prove that such a construction method will also be safely applicable for bridge piers in these days.

In the preliminary report of this Congress you find under theme IVa a paper of our French engineer colleagues M. Gerbault and P. Xercavins informing us that the Shell platform CORMORANT will be founded in the North Sea at a place where the water is 154 m deep. Compared to these gigantic structures our bridge piers are small. But the large height of the foundation tower suggests to use ring foundation similar to those which I developed for large TV towers. The proposed ring foundation will cut down the necessary amount of concrete and steel considerably and will result in great economy compared to conventional types of foundation. I would appreciate if this proposal would be considered for the big bridges crossing sea straits, which are being planned in several parts of the world.

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Preliminary Main Pier Designs for the Great Belt Bridge

Avant-projet des piles principales du pont sur le Great Belt

Vorstudien für die Pfeiler der Brücke über den Grossen Belt

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The design of the main piers for the Great Belt Bridge has during the last decade been investigated in a number of studies.

In the chosen bridge alignment the water depth reaches 55 m, and a direct foundation can be established on Danian Limestone or Paleaoic Marl few meters below.

This paper describes shortly three pier designs, here designated pier A, B and C.

PIER A

For the International Design Competition 1966 a road bridge as shown in figure 1, with a main pier design as shown in figures 2 and 3 was proposed by the author et al [1].

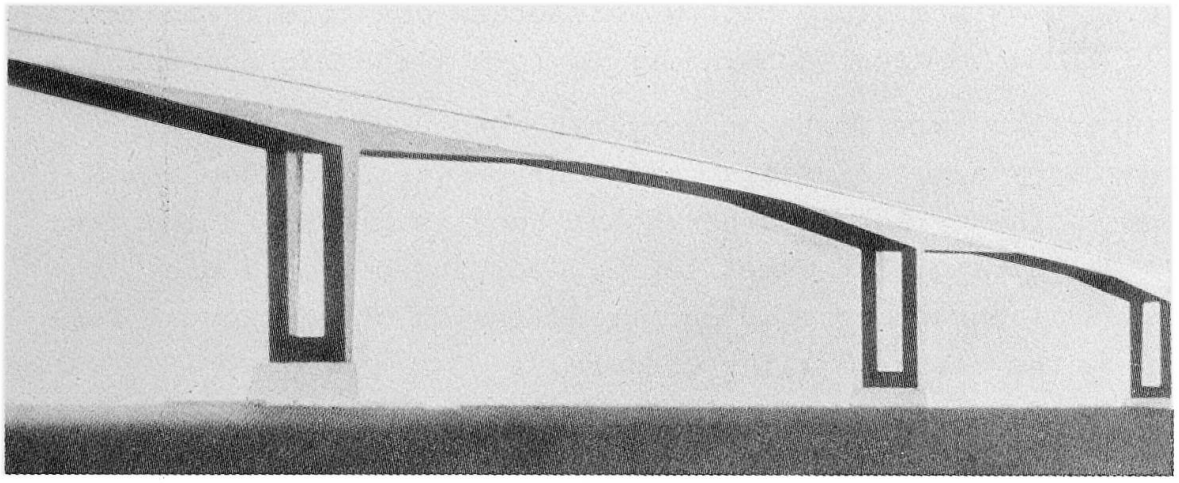


Figure 1. Road bridge designed as a frame with two 400 m wide main spans. From a first prize design by Gimsing, Madsen and Nissen in the International Design Competition 1966.

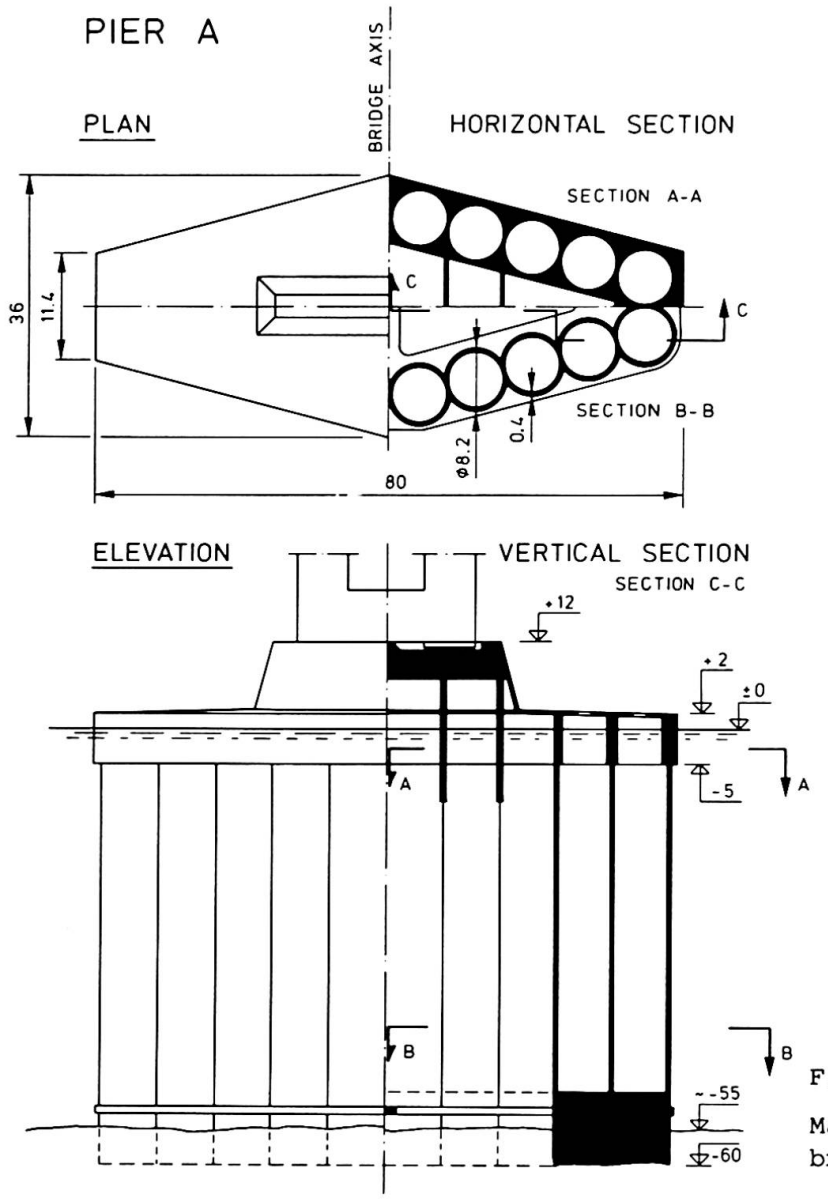


Figure 2.

Main pier design for the bridge shown in figure 1.

This pier, founded 60 m below water level, should be designed for very large forces from ice pressures and collision (equivalent static collision force: 120 MN).

The pier consists of 20 hollow reinforced concrete cylinders joined along vertical generators to form a rhombic box section.

The superstructure rests on a plinth connected to the wall of cylinders by a covering slab and four prestressed deep beams. On top of the cylinders a hexagonal hood is placed to increase the local strength at water level. Near the bottom a prestressed diaphragm assures the transmission of horizontal forces from the earth to all cylinders.

The construction of the pier should start in a dry dock by casting the lower part. After placing of temporary bulkheads

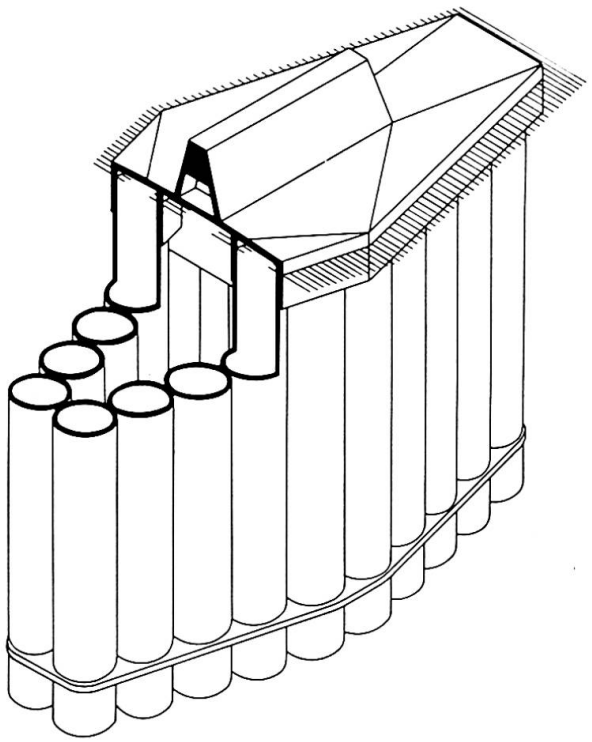


Figure 3.

Isometric projection of the pier design shown in figure 2.

the pier can be floated to deeper water and the remaining part cast in slip forming. When placed in final position the temporary bulkheads are removed and excavation inside cylinders carried out. Finally, concrete plugs are cast under water at the bottom of all cylinders.

PIER B

1970-72 a Technical Commission appointed by the Ministry of Public Works made a comparative study of different solutions for the Great Belt crossing, among these the cable-stayed bridge shown in figure 4. [2].

Here a more solid pier structure, as shown in figure 5, was proposed. The pier is composed of square wells, 42 in the lower and 20 in the upper part.

In principle this pier should be constructred as pier A, with the exception that temporary steel tubes had to be added to prolong the outer wells during floating and excavating.

A dynamic investigation of the collision case gave the following results:

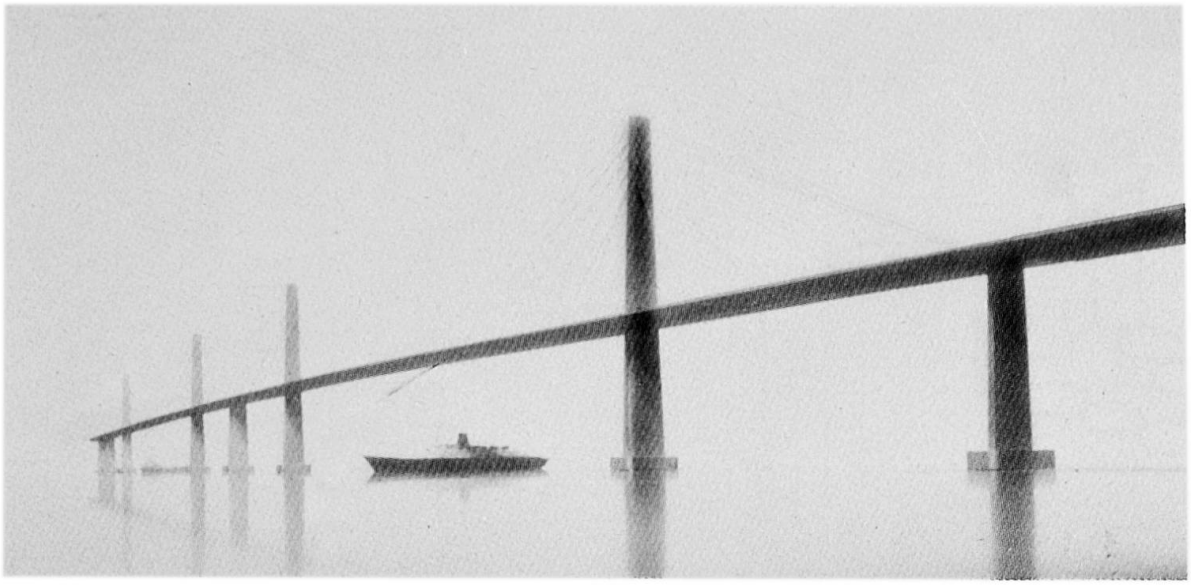


Figure 4.

Combined road and rail bridge designed as a cable-stayed bridge with two 600 m wide main spans. From the investigations made by the Technical Commission 1971.

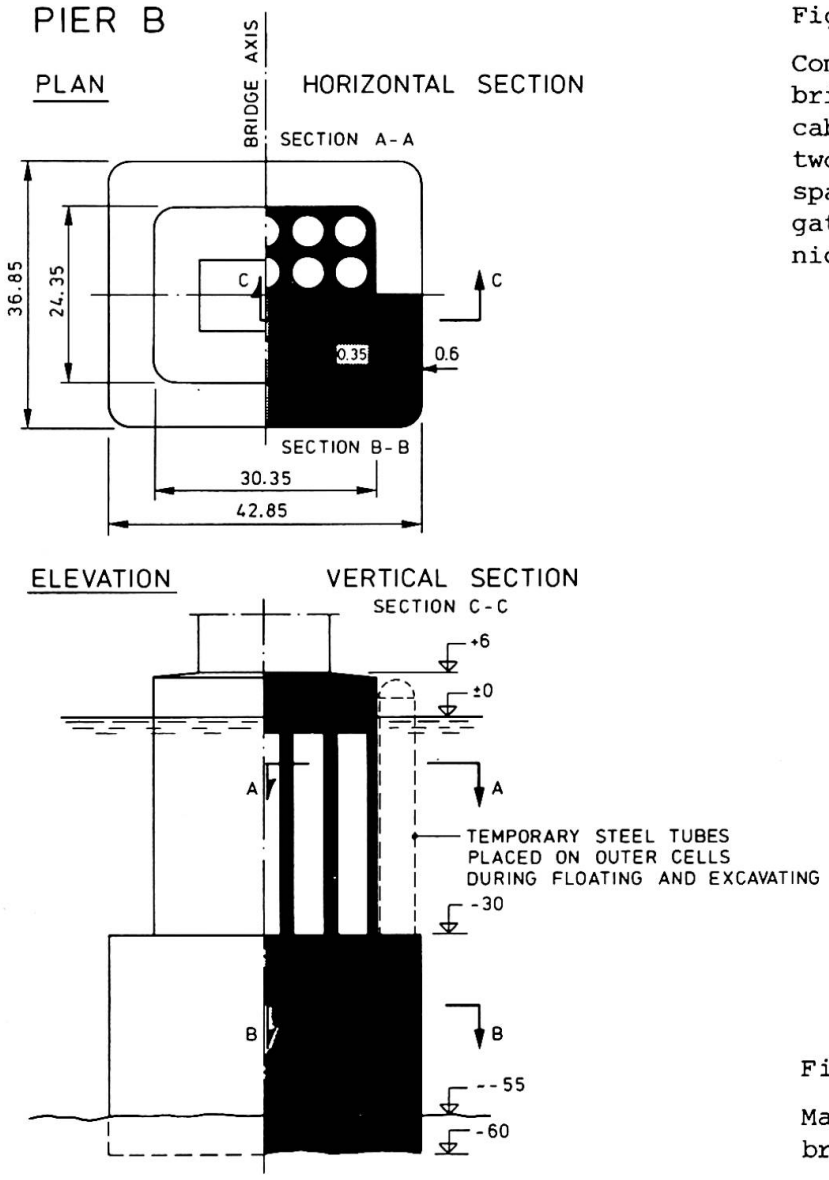


Figure 5.

Main pier design for the bridge shown i figure 4.

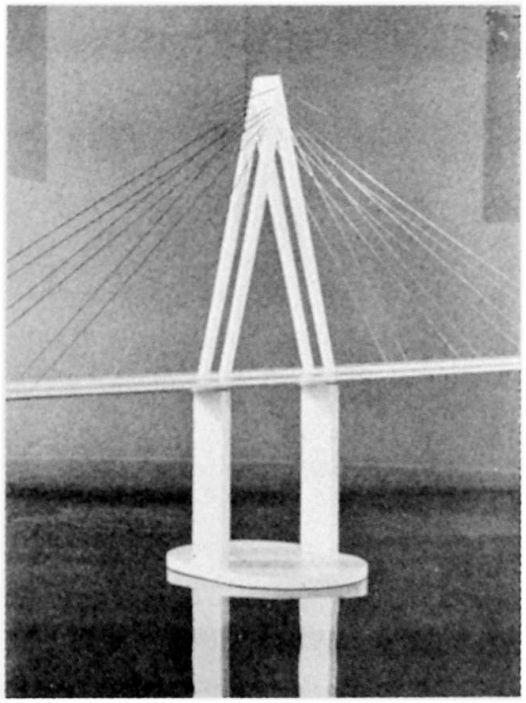
A collision by a ship of maximum 14.000 tdw going at full speed, or a tanker of 100.000 tdw drifting abeam at current speed would only give local damage.

A collision by a tanker of 100.000 tdw going at full speed will give a permanent horizontal displacement of 0.1 m at water level. In the superstructure this displacement only induces a flexural stress of 90 MPa and consequently no danger of collapse would exist.

PIER C

Recently, the author has made a study of a cable-stayed bridge with two main spans separated by a triangular pylon structure (figure 6).

The corresponding pier structure, shown in figure 7, contain a wall of cylinders as in pier A, but the number is increased to 26 due to the larger width required to give support for the double pier shafts.



Despite the significantly larger outer dimensions pier C requires only slightly larger quantities than pier B. This is due to the hollow design, which also gives an advantageous increase of the stability.

Figure 6.

Triangular pylon structure investigated by the author 1975.

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[1] Gimsing, N.J., K. Madsen and J. Nissen: Great Belt Bridge. Väg- och Vattenbyggaren. No. 5, 1967.

[2] Gimsing, N.J.: Pont sur le Grand-Belt (Danemark), Travaux 1973 no. 463.

[3] Leonhardt, F. & W. Zellner: Cable-stayed Bridges: Report on latest developments. Canadian Structural Engineering Conference, 1970.

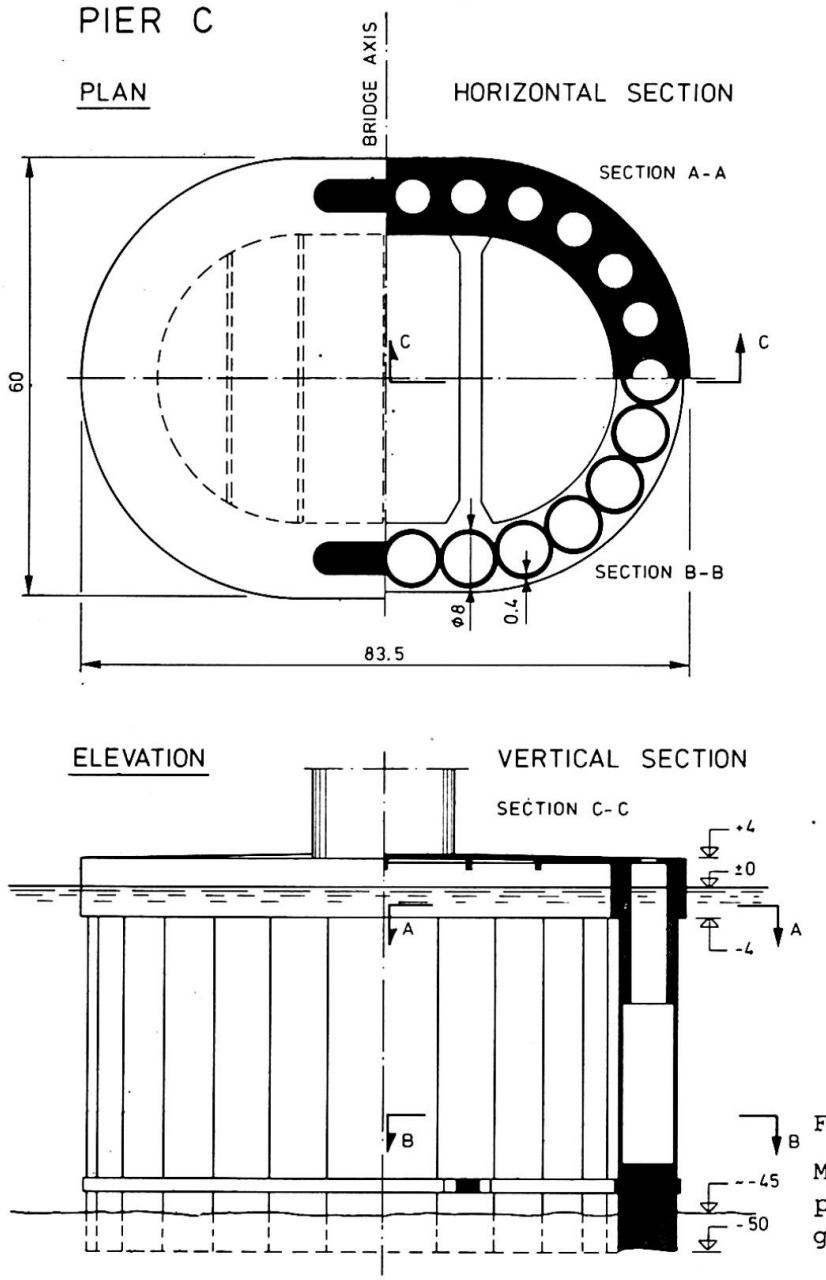

Figure 7.
Main pier design for the pylon structure of figure 6.

[4] Leonhardt, F.: Fundationen für weitgespannte Brücken, Einführungsbericht, IVBH, 10. Kongress, Tokyo 1976.

[5] Gimsing, N.J.: Multispan Stayed Girder Bridges. ASCE Journal of the Structural Division, October 1976.

SUMMARY - Preliminary investigations have indicated that the main piers for the Great Belt Bridge can advantageously be constructed by using hollow concrete caissons.

RESUME - Les études préliminaires ont montré que les piles principales du pont sur le Great Belt pouvaient être avantageusement réalisées à l'aide de caissons creux en béton.

ZUSAMMENFASSUNG - Vorstudien für die Pfeiler der Brücke über den Grossen Belt haben gezeigt, dass Caissons aus Beton Vorteile bieten.

Foundations of Zarate-Brazo Largo Bridges

Les fondations des ponts Zarate-Brazo Largo

Die Gründung der Zarate-Brazo Largo Brücken

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1 - Introduction

The Zárate-Brazo Largo Highway-Railway Complex, now under construction, will permit the crossing of the Paraná River at about 80 km North of Buenos Aires City. At present this crossing is made using rafts and ferry-boats, and these bridges will permit a great saving in time and cost, not only for the Argentine interprovincial traffic but also for the international traffic with Uruguay and Brasil.

At this point, the Paraná River is divided in two branches, each navigated by ocean going ships. The Complex is, then, formed by two identical bridges with a free span and clearance of 300 m x 45 m respectively. The bridges' decks support one railroad and one four lanes highway.

Each bridge is 550 m long, with a central span of 330 m and two lateral spans of 110 m each. Both are cable-stayed bridges with cables fanning out from the top of the 120 m high towers. These towers, as well as the two anchorage pier, are of reinforced concrete hollow rectangular section. The deck is of high resistance steel, and is formed by two lateral box girders and plate deck, suitably stiffened and covered with a reinforced concrete pavement.

2 - Description of Foundations

Both bridges' main piers (towers) and the two anchorage piers of one of them are on the river bed. The other two anchorage piers, and almost every access viaduct pier, are on the flood bed of the Paraná River.

The soil over the entire width of the river flood bed is formed by alluvial deposits of great depth. The upper layers show lime and soft clay. At different depths there are some half consolidated clay layers, sand layers, and finally very thick dense sand layers. These dense sand layers can take the loads transmitted by the piers. The upper part of these sand layers is at depths ranging between 20 and 50 m.

Consequently, all piers were founded on big diameter piles. Two-meter diameter cast in place piles were used, with steel casing when built in water, and without steel casing when built on land.

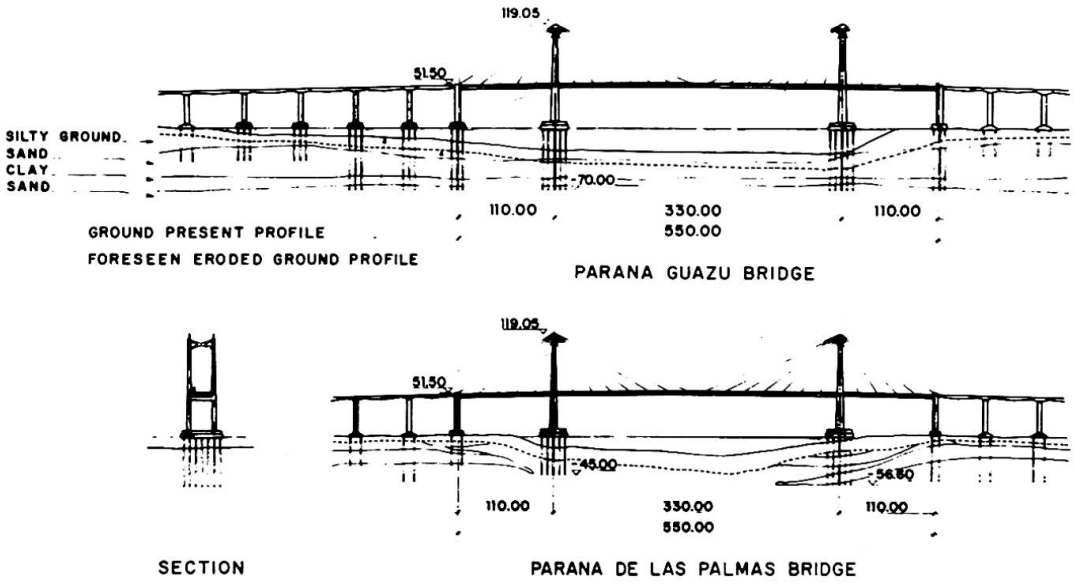

Fig. 1 - Bridges lateral views and geological profiles

The piles' length was determined by the need of entering a minimum of 5 m in dense sands. Also the eventual future modifications of the river beds due to erosion phenomena both general and around the piles, have to be taken into account in said determination. Thus, pile length varied between 25 and 73 m.

Parallelepiped pile caps, lightened by hollows, join the piles. These pile caps are built over the water level. Preformed slabs hang from their lower edge and sink some centimeters into the water for aesthetic reasons.

Based on the soil characteristics and taking into account load transmission by the tip and lateral friction, the maximum allowable vertical load for piles in water was established at 1200 tons plus the pile weight.

A simple arrangement was adopted for the plant distribution of piles. They were lined up in both directions, separated by a distance practically the same as the minimum advised (three times the pile diameter). Because of the different water depth in both rivers, now and even more in the eventual future situation of an eroded bed, a greater number of piles was needed for the Paraná Guazú than for the Paraná de las Palmas. Because of its greater slenderness and greater bending moments due to horizontal loads, the maximum allowable vertical load on the pile head had to be reduced in the Paraná Guazú piles to 1000 tons.

For the Paraná Guazú main piers, 45 piles and one parallelepiped pile cap were adopted. The pile cap has a plant of 30 m x 59 m, and a thickness of 5,50 m and is lightened by hollows in the less stressed areas. The concrete volume for this cap was 8,300 m3, reinforced with 1,100 tons of high strength steel.

For the Paraná de las Palmas, 33 piles and one pile cap, with an "I" type plant of 30 m x 45 m and a thickness of 5,50 m, also lightened, were adopted. The concrete volume was 6050 m3, reinforced with 860 tons of high strength steel.

3 Pile-caps Calculations

Pile cap as a whole is a thick slab bearing on a series of point supports. The load on this slab is applied on rectangular areas noticeably smaller than the slab's dimensions. This slab is lightened by cubic holes in areas far from the load application area. Thus, it might be considered as a grid, supported on every point of girder crossing. However, the girders are joined by very thick slabs, in their upper and lower parts, which increase the grid stiffness considerably.

Hence, we performed a double calculation following two different approaches:

The first one was to use a grid model, solving it by STRUDL. Loads on piles and values for bending moments and shear forces on the different members were thus obtained and the necessary reinforcing steel sections for the different parts of the pile cap was determined.

The second approach was to try to follow the stress paths into the mass of pile cap accepting the same loads on piles as obtained by the first procedure. Concrete stresses were verified and the necessary steel sections for tensioned members were determined. These were compared with those steel sections calculated before, and the bigger one was selected in each case.

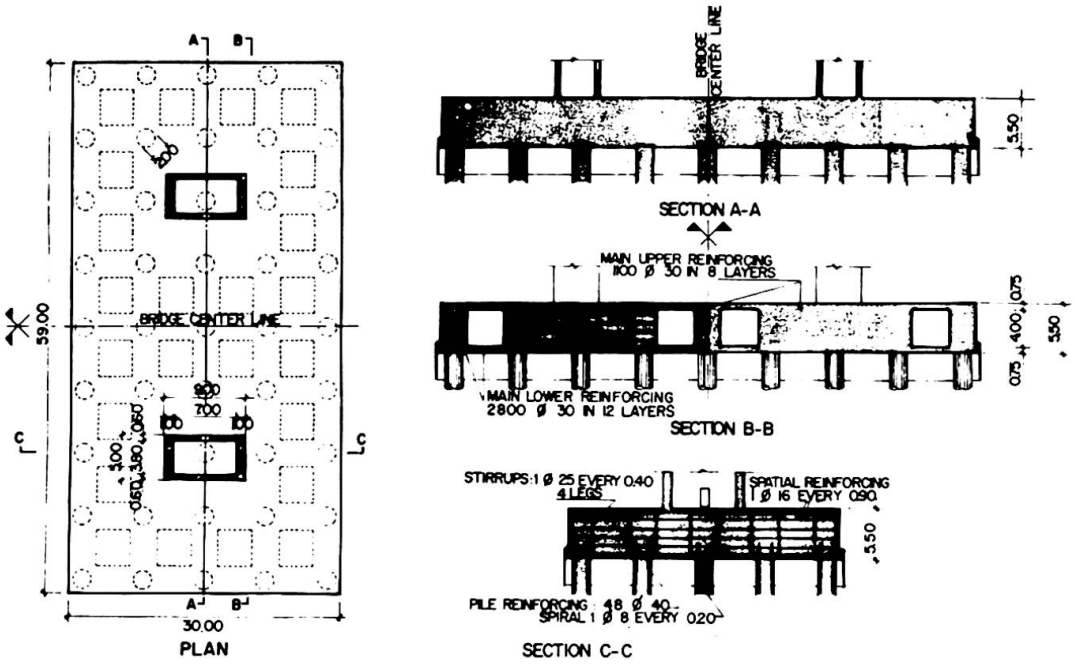

Fig. 2 - Paraná Guazú Main Piers Pile Caps

4 - Piles Calculations

Two questions arose from the beginning of the project regarding dimensioning and verification of piles in water:

a) How to determine the actual pile embedding into the soil, in order to establish buckling length and bending due to horizontal loads.

b) How to consider the pile cross section to take into account steel casing.

Because of scheduling it was impossible to perform horizontal load test in time to use its results for design. Thus, the Winkler elastic model of the pile-soil system was adopted. More sophisticated models, i.e. considering the pile as an elastic element sunk into a semi-infinite elastic medium, or taking into account the soil plasticity and viscosity were discarded due to the lack of enough experimental data.

According to the adopted model, it is acceptable to consider soil horizontal reaction P, at a certain depth x, proportional to the horizontal displacement of this point of the pile, that is: $P = Es \cdot y$

Factor Es is the soil reaction module, which generally varies with depth. Taking into account soil characteristics, it was decided to adopt a parabolic relationship, as follows: $Es = 50 \sqrt{z}$ (z expressed in cm; Es in kg/cm2).

Once the supposed elastic fixing characteristics were thus defined, the solution of the structural problem could be obtained replacing this elastic fixing by a rigid fixing at a certain depth $\Delta\ell$ below the soil level. To obtain this depth, we calculated the length of a pile rigidly fixed at both ends, with the same cross section as the actual pile, which under an horizontal load on its head will deflect the same as the pile perfectly fixed in the pile cap and elastically fixed in the soil would do. In this case, we obtained $\Delta\ell = 7.00$ m.

As for the second question, the problem was to decide if the structural member obtained by filling a steel tube with concrete can or cannot be considered equivalent to a reinforced concrete column.

Several publications mention theoretical studies and test made about this problem. However, there is still no complete treatment which allows the establishment of dimensioning rules, which could be used in our case, mainly because the large diameter.

The following criterion was finally adopted: first, calculate the maximum allowable axial load without bending, using the ACI formula for tubular steel columns filled with concrete; then calculate the maximum allowable bending moment, without axial effort, considering the pile as a reinforced concrete column using Jimenez Montoya's ultimate strength charts.

As the reinforcement for this column we computed the complementary inner reinforcing made up by common steel round bars, with a spiral tie, plus the full useful section of the common steel casing (that is, after deducting 5 mm for corrosion allowance).

Thus established the two end points of the interaction curve M - N , this curve should be determined. The linear relationship joining such points can be considered as a lower limit. Taking into account the convex shape of interaction curves for reinforced concrete columns ultimate strength calculations, we considered our curve could follow a quadratic relationship.

Nevertheless, later comparisons with the recommendations of CIDECT (Comité International pour le Développement et l'Etude de la Construction Tubulaire) indicated the convenience of introducing a correction, reducing the allowable moments in the zone of maximum axial load.

Thus, an elliptical interaction curve was adopted, the ellipse's two semiaxes being given values of N_{all} for $M = 0$ and M_{all} for $N = 0$. Where there were high axial force values, the elliptical relationship was replaced by a linear relationship obtained by plotting the line by the two points (N_{max}; $M = 0$) and ($N = 0$; $2\,M_{max}$).

One detail of the design that merited special attention was the linking of the pile with the pile cap. In the static scheme adopted the piles are fixed in the pile cap. Normally, this type of fixing is materialized by lengthening the reinforcing of the entire pile and anchoring it into the pile cap. In this case, a very important part of the pile's reinforcing is represented by the casing. However, it was totally impractical to lengthen the casing in the pile cap and anchor it in such a manner that the casing's maximum tension load could be transmitted to the pile cap, because it would have produced considerable interferences with the reinforcings of the pile cap.

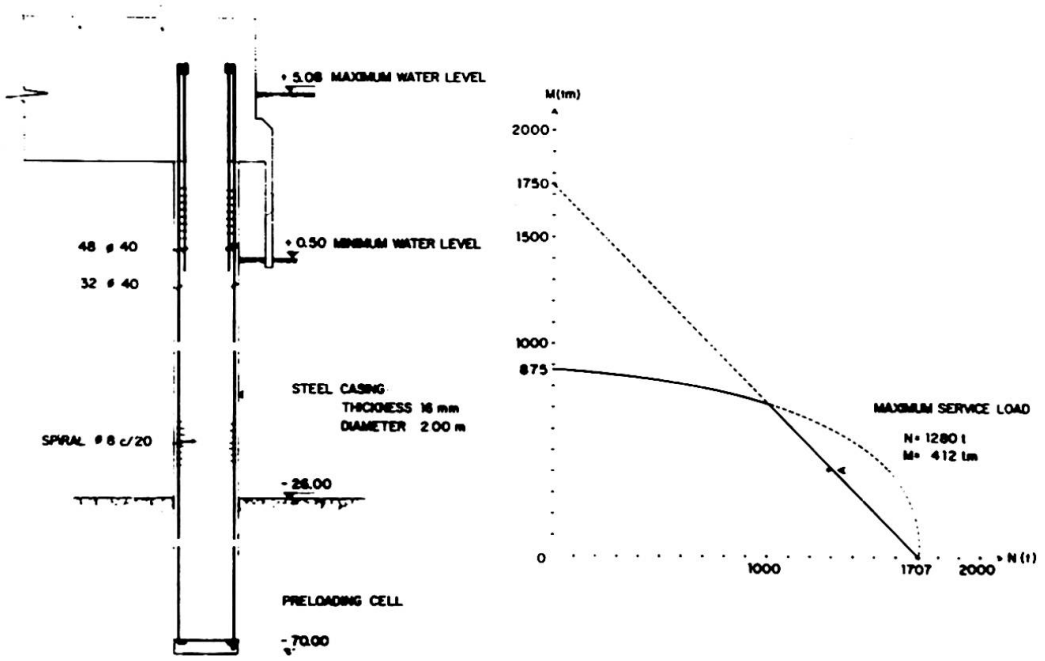

Fig. 3 - Pile Details and Interaction Curve

On the other hand, it must be taken into account that the vertical force on the pile is always compression and that tension could only happen in a very limited sector of the casing under exceptional load conditions, and with very low tension stresses. Thus, the idea of anchoring the casing to withstand tension forces was discarded, adopting instead the transference of the compression force by installing a collar of thick steel plates that provides a contact surface with the concrete ten times greater than the casing's effective section.

On the contrary, about the complementary inner reinforcing it was decided to lengthen it within the pile cap with a sufficient anchor length. In addition, to parcially compensate the discontinuity that occurs in the casing interruption, the complementary reinforcing in the section near and into the pile cap was increased 50 %.

5 - Piles Construction

The equipment used consisted of a floating barge with two cranes, one with a 75 tons capacity plus an excavating attachment, and the other, used for auxiliary service, with a 25 tons capacity. In addition, a hydraulic claw permits the handling of the steel casings.

After securing the barge in the exact position, a temporary steel casing slightly larger than the pile was lowered, penetrating some 5 meters below the river bed.

Then, the hole is excavated by means of a rotating bucket. During the excavation process, both perforation and temporary casing are permanently filled with bentonitic slurry.

Once the perforation reached the prescribed depth, the steel casing, 16 mm thick, is placed. The different parts of the casing are welded together, while they are driven downward, by means of x rays controlled welding. At this point, the temporary casing in the upper part was no longer necessary and it was removed by the claw. Then, the slurry was replaced by clean water and the placing of the pre-loading cell and reinforcing proceeded.

After concreting the pile under water, the job is completed by cutting the casing at the predetermined spot and welding the supporting collar. Finally, the pre-load grouting is performed.

The pre-loading cell consists of a metallic basket filled with crashed stone. After concreting the pile, cement grouting, under pressure of up to 80 kg/cm2, is inserted into the cell by means of a system of tubes and valves.

With this procedure two beneficial effects are obtained for the pile's load capacity: to fill any eventual cavity or area of loose soil that may exist close to the pile tip; and to compress the lower soil with a total load of 2500 tons. Under this load, which approximately doubles the working load, it is reasonable to expect that complete soil settlement is occured in such way that under working loads the pile cap's settlement is due only to the elastic shortening of the pile.

6 - Pile Cap Construction

The majority of the pile caps for main bridges' piers are located in the river bed; thus, the form bottom was supported by a temporary steel structure that rest on provisional brackets welded to the pile casings.

Steel forms are used for the bottom and sides of the pile cap. Concreting is done in several stages planned to simultaneously minimize the shrinkage effects and obtain a good monolithic structure. Thus, the working joints are offset in the different layers and have a serrated surface.

The concrete specified for these pile caps has characteristic strength of 210 kg/cm2, which has been tested with ample margin. The reinforcings use type III steel (cold twisted) with an allowable stress of 2.400 kg/cm2.

SUMMARY

Foundations of two identical bridges of 330 m span over the Parana River include large diameter steel-cased cast-in-place reinforced concrete piles and reinforced concrete pile caps. This paper describes design criteria and construction methods used for these foundations. Due to great diameter and slenderness of piles, special formulae to verify safety conditions were used.

RESUME

Les fondations de deux ponts identiques de 330 m de portée sur la rivière Parana sont composées de pieux de grand diamètre bétonnés sur place dans des tubes en acier sur lesquels reposent les semelles en béton armé. Ce rapport décrit la méthode de calcul et les techniques de construction utilisées. Dû au grand diamètre et à l'élancement remarquable des pieux, des formules spéciales ont été utilisées pour vérifier la sécurité.

ZUSAMMENFASSUNG

Die Fundation der zwei gleichen Schrägseilbrücken über den Parana mit einer Hauptspannweite von 330 m besteht aus Stahlbeton-Pfählen grossen Durchmessers mit einer äusseren mittragenden Stahlhülse und verstärkten Pfahlkopfplatten. Dieser Artikel beschreibt das angewandte Berechnungsverfahren sowie Baumethoden. Wegen des grossen Durchmessers und der grossen Schlankheit der Pfähle, wurden spezielle Formeln angewendet, um die Sicherheit nachzuweisen.

Humber Suspension Bridge South Tower Caisson Foundations

Fondations en caissons de la tour sud du pont suspendu sur le Humber

Die Senkkasten-Fundation des südlichen Pylons der Humber-Hängebrücke

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The Humber Suspension Bridge, which will have the world's longest main span - 1410m - is situated about 300 km north of London, England. On the north side of the river, the bridge foundations presented relatively few problems, being located in chalk. The foundations on the south side, however, are located in Kimmeridge clay overlain by glacial and alluvial material, posing considerable design and construction problems.

Kimmeridge clay is an over-consolidated fissured silty clay. It is hard under the existing over-burden pressure but rapidly disintegrates under the action of water when this pressure is removed. The basis of the foundation design for both the south anchorage and south tower was therefore to minimise the heave of the clay and so keep the fissures closed to prevent the ingress of water.

The Consulting Engineers prepared two alternative designs for the tower pier foundations, one using large diameter bored piles, the other using caissons. Messrs John Howard, the Contractor, who constructed all the bridge foundations, submitted a lower price for the caisson scheme than for the piles.

The tower pier is located about 500m from the south bank of the river in water about 8m deep at high tide. The twin hollow circular caissons, each 24m dia, were sunk through an artificial "sand island". The level of the sand island top was about +4m and the founding level of the caisson was -36m, giving a penetration of about 7m into the Kimmeridge clay.

Caisson sinking was by excavating under water using grabs, since the founding level was too deep to permit the use of compressed air. To control verticality of sinking and prevent base heave, the lower 27m of each caisson was divided into one centre cell and six outer cells. Thus, after de-watering the caissons, bottoming and plugging could be carried out one cell at a time, if necessary, counterweighting by sand fill all but one unplugged cell.

Calculations based on shear box tests on consolidated samples of 10% bentonite suspension, indicated a skin friction of about 20kN/m^2 for sinking the caisson to -25m through sand. "Actual" skin friction values (using the results of divers' surveys to assess the end bearing of the caisson) were estimated as 25-28kN/m^2. A sinking effort of 28kN/m^2 was found to be satisfactory. Calculations estimated average skin friction to be approximately 40kN/m^2 if no bentonite were employed.

In sinking the caissons, considerable difficulty was experienced in the cohesive material. The only satisfactory way of removing the clay from beneath the haunched cutting edge was by means of high-pressure water jets, working at about 30kN/m^2, either lowered down holes in the caisson walls, or held in purpose-built frames in the cells. A sinking effort of 50kN/m^2, including kentledge proved necessary to sink in the Kimmeridge. This high figure was due to partial destruction of the bentonite annulus arising from difficulties encountered in the overlying glacial and alluvial material.

During de-watering and final bottoming, borehole extensometers were used to check the amount of heave below the foundation and give warning (although there was a time lag between the removal of load and full recovery), thus permitting the envisaged sand counterweighting procedure mentioned above to be dispensed with and saving considerable time thereby. After plugging, the caissons were ballasted with water to prevent generalised heave of the foundation area. The water was removed as the external applied load was increased by construction of the concrete tower.

SUMMARY

The foundations for the south tower of the Humber suspension bridge (1410 m main span) are in Kimmeridge clay overlain by glacial and alluvial deposits. Twin hollow caissons 24m dia divided into one centre and six outer cells sunk to a depth of 40m through a sand island in the river by use of grabs, high-pressure water jets and kentledge, the penetration into the clay being 7m. A sinking effort of 50kN/m^2 was required. After plugging the caissons were loaded with water, which was removed as the load of the tower increased during its construction.

RESUME

Les fondations de la tour sud du pont suspendu sur le Humber (portée centrale 1410m) sont fondées dans l'argile de Kimmeridge, située au-dessous des sédiments glaciaires. Des caissons jumelés creux de 24m de diamètre, formés d'une cellule centrale et de six cellules extérieures, étaient enfoncés, dans un îlot de sable construit dans la rivière, jusqu'à une profondeur de 40 m. On a lesté les caissons et excavé à l'intérieur avec des bennes et des jets d'eau à haute pression pour pénétrer de 7m dans l'argile. La pression d'enfoncement nécessaire était de 50 kN/m^2. Pour augmenter le poids, les caissons étaient d'abord bouchés et chargés d'eau. Puis, l'eau était évacuée à fur et à mesure que la construction avançait.

ZUSAMMENFASSUNG

Die Fundamente des südlichen Pylons der Humber Hängebrücke (Mittelöffnung 1410 m) stehen im sog. Kimmeridge Lehm, welcher von Gletschergeröll und alluvialen Anschwemmungen bedeckt ist. Hohle doppelte Betonsenkkästen von 24m Durchmesser, unterteilt in eine Mittel- und sechs Aussenzellen, wurden durch eine Sandinsel im Fluss mit Hilfe eines Greifbaggers, Wasserdruck und zusätzlicher Belastung 40m tief, davon 7m in den Lehm, versenkt. Eine Auflast von 50kN/m^2 war hierzu nötig. Nach Abdichtung der Senkkästen wurden sie mit Wasser gefüllt, welches während der Herstellung des Pylons sukzessive wieder entfernt wurde.

Foundation of a Reinforced Concrete Arch Bridge

Fondations d'un pont en arc, en béton armé

Fundation einer Stahlbeton-Bogenbrücke

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FOUNDATION OF THE REINFORCED CONCRETE ARCH THE SPAN OF 390 M

The bridge connecting the Yugoslav Coast with the Island of Krk, in the Adriatic Sea, consists of two reinforced concrete arches over which a prestressed concrete superstructure is to be installed.The appearance of the bridge is shown in Fig.1.

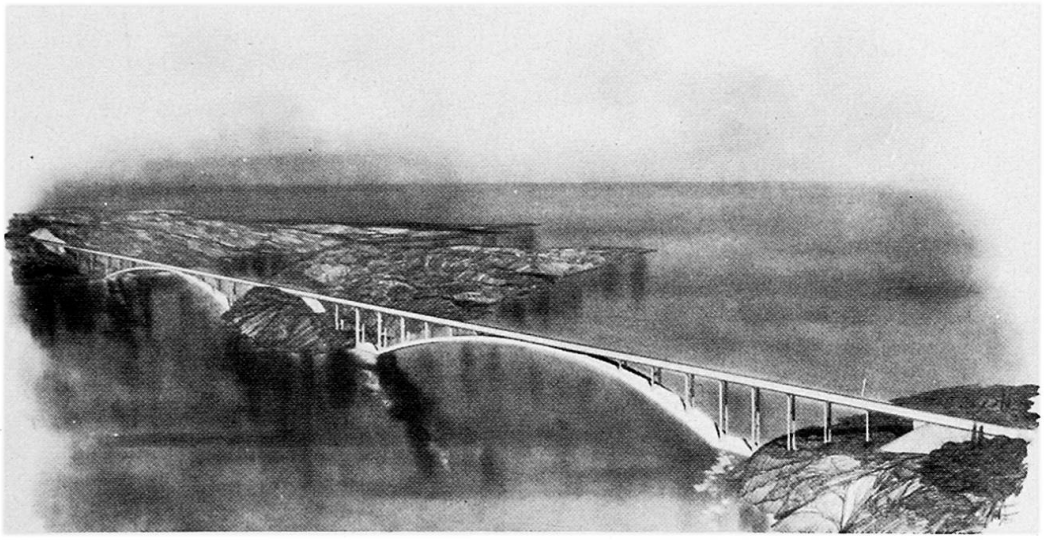

FIG. 1

The reinforced concrete arch over the smaller opening has the span of 244 m, and the arch over the larger one, has 390 m between two supports.As the distance between two shores in this opening is much longer, over 460 m, two arch supports shall be partially founded in the sea,in order of reducing a span,as shown in Fig.2 .

The arch reaction (of approximately 14.000 t) has been distributed to the filled oblique pier founded in the sea (transferring 9.000 t to the rock),and to the nearly horizontal box structure above the sea level that transfers to the coast rock mass the other component of the arch reaction (6.000 t),and the arch bending moment originated from the bridge loading.

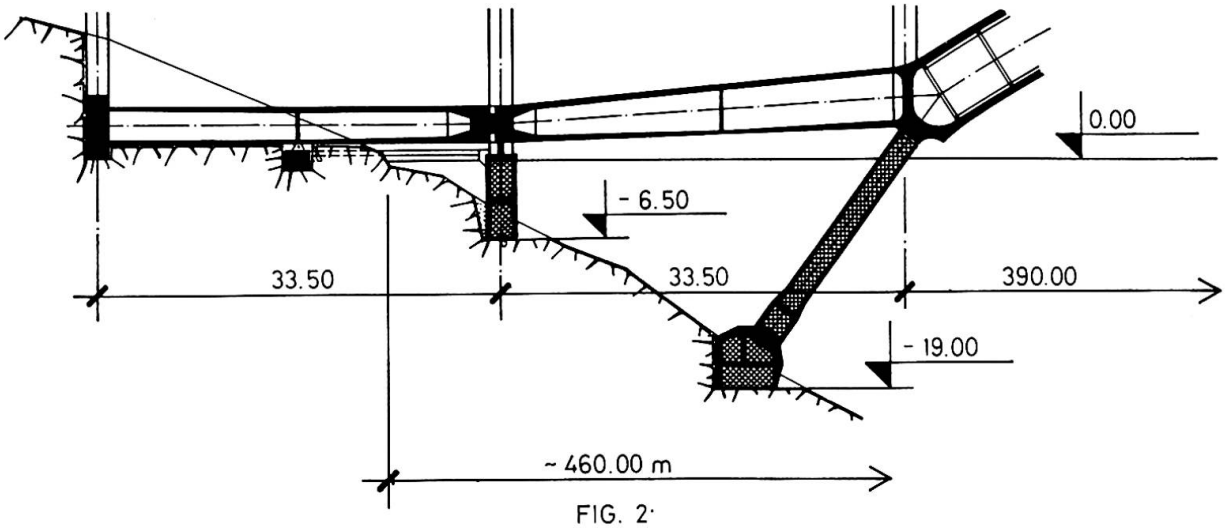

FIG. 2·

The bridge construction works are being performed by "The Civil Engineering Enterprise Mostogradnja " from Beograd, Yugoslavia,according to its design.

The process of foundation works shall be revealed through the construction stages.

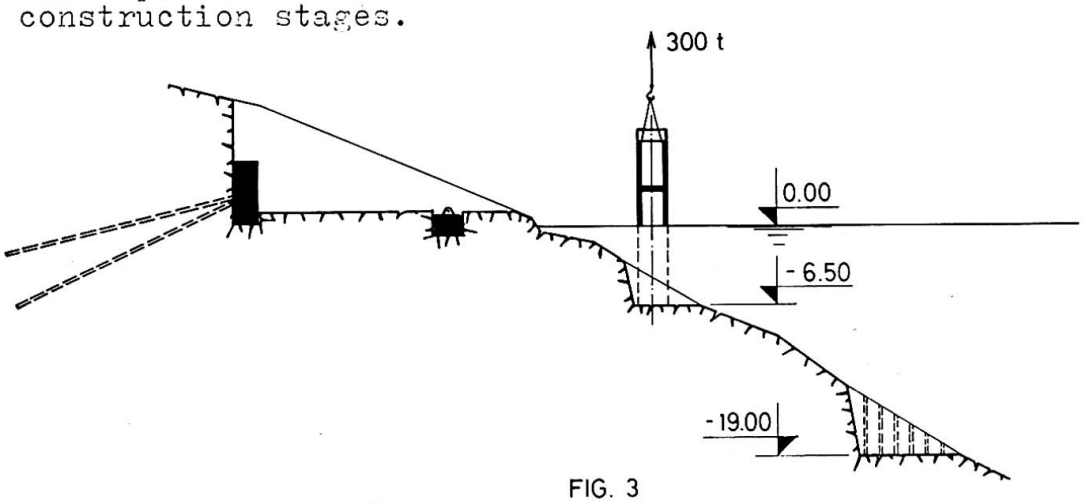

FIG. 3

To mine and excavate the limestone at the places of the arch foundation,was the first step to be executed (see Fig. 3.). The sea boreholes have been prepared operating from a floating device and the explosive installed by the divers. The excavation of material has been executed by the excavators. Finally, a team of divers has accomplished the leveling and clearing operation of the foundation bottom at the level of -6,5o, i.e. -19,oo m under the sea surface, performing corrective mining where it was necessary.

Upon completion of excavation on the shore the anchors shall be fixed into the rock, in the lenght of 2o,oo m. The cables consisting of 24 steel wire cords, ⌀ 7 mm,shall be installed in the bores,diameter of 1o5 mm. The entire anchor force of 3800 t shall be then applied to concreted lateral footing.

Simultaneously to the excavation,affixing the anchors into the rock and cocreting of the foundation onthe shore,there shall be prepared a hollow pier structure devided into the appropriate compartements,with a caisson chamber in the bottom part. Concreting is to be executed at the site auxiliary quay where it shall be transferred from,to the sea,by a floating crane, the bearing capacity of 36O t, and installed in the bottom.

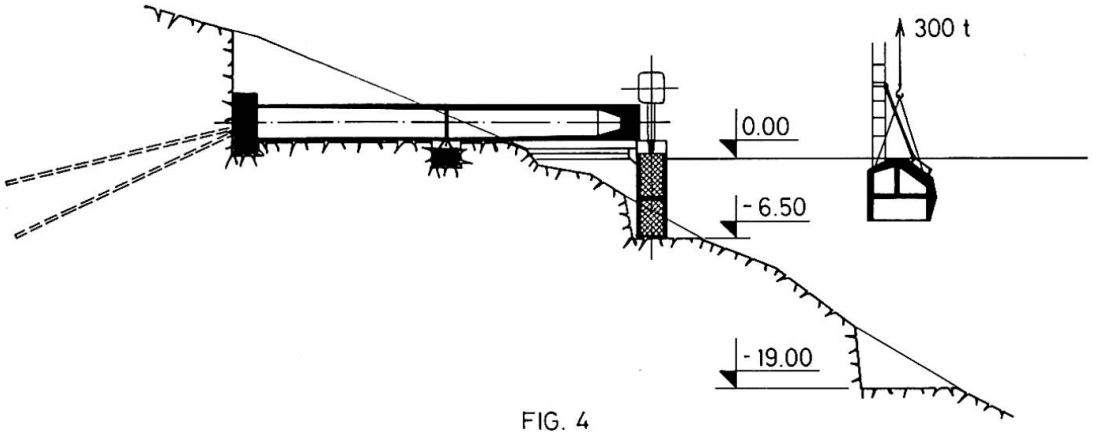

FIG. 4

When placing in its position, the pier shall be connected to the foundation on the shore by a couple of prefabricated concrete ties,thus determining its precise position. Having the protection of compressed air the level of the pier has been regulated by four hydraulic jacks installed in the corners of the caisson chamber.

Concreting of the hollow pier compartments has been planned to be performed upon completion of clearing the soil in the caisson bell (see Fig. 4.).Providing the protection of compressed air,the first step was to concrete in a caisson bell and subsequently the upper part of the pier,performing the concreting operation in waterless space having the water previously exhausted.

Concreting of the horizontal box structure, extending from the pier to the foundation block on the shore, shall be accomplished simultaneously to the above described.This structure shall be prepared, by prestressing,to accept and transfer to the anchors,which are fixed in the rock, the tensile stress occuring in it, prior to assembling of the reinforced cocrete arch.

Together with these works the caisson bells for the arch oblique pier foundation shall be concreted.This shall be performed on a floating platform provided with self-sinking and floating mechanism. The bells should be lowered until the entire body is immersed as their original weight of 300 t, shall then be considerably reduced (up to 140 t, due to existing air in the upper compartments),thus facilitating their transfer and placement by the crane to the previously prepared ground at the level of -19,oo m from the sea surface.

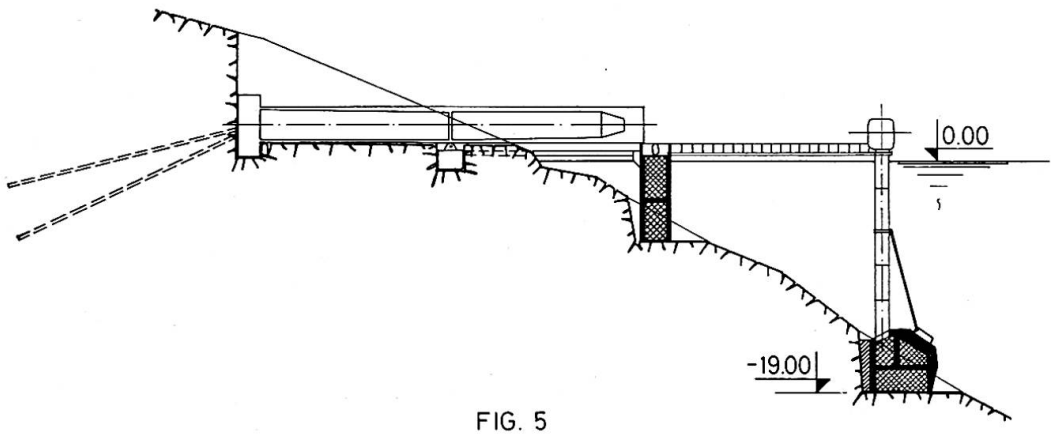

FIG. 5

The caisson bell should be placed on the bottom along with a pair of caisson tubes (see Fig. 5.). These tubes are coupled transversely and their longitudinal position affixed by the obliquely arranged steel bars and by the steel structure of the approaching bridges, thus affixing the position of the caisson bell itself. The foundation level is controlled by the hydraulic jacks installed in the corners of the caisson chamber.

Completion of excavation intended to level and clear the terrain underneath the caisson bell should be followed by concreting of the foundation cavity. The chamber is to be filled with concrete under the protection of compressed air and the upper compartments through the caisson tubes as the water has been previously pumped out from those and the hardening of the concrete in the lower chamber has progressed sufficiently.

The hollowed area next to the caisson, toward the rock mass, should be filled with concrete by the application of the contraction method, for the purpose of confronting with the horizontal component of reaction originated from the arch oblique pier.

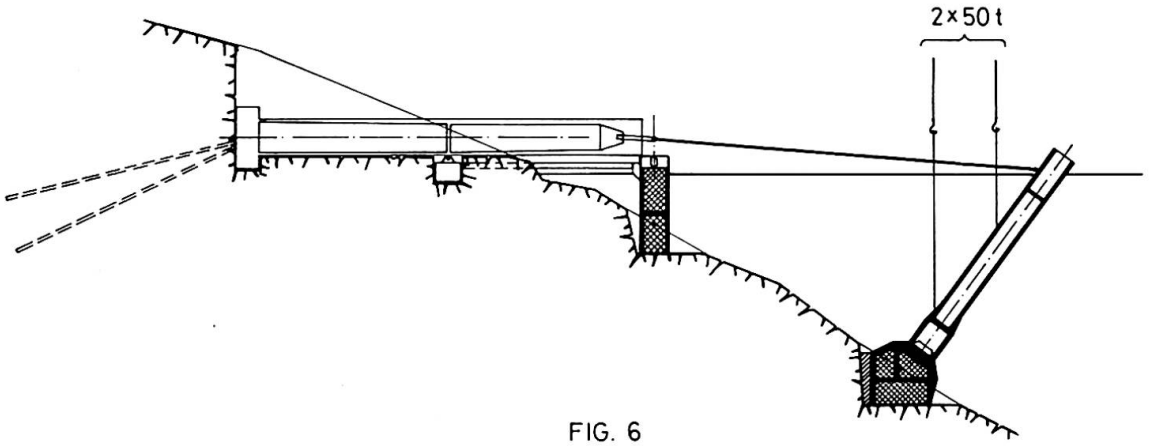

FIG. 6

At the same time the excavation in the caisson bell is being performed, followed by the pouring of concrete into the oblique pier foundation cavity, the oblique pier box structures should be as well concreted, on the same floating platform upon its floating out to the surface. By sinking the platform the oblique piers should be transferred by the floating crane to their determined positions (see Fig. 6.). The cavities in the pier penetrated by air are designed in a manner to reduce its weight, after being immersed, to the rate suitable for the installation operation. Affixing of the oblique pier into,by the design predicted position, should be accomplished attaching the temporary steel ties equiped with the device for the adjustmeht of their length.

Upon sealing the joint of the pier bottom and foundation by the application of the epoxy resin,concreting of the pier hollow body should be commenced in waterless conditions. The chamber next to the foundation is the first to be filled with concrete, and then, one by one, the other compartments after the water has been pumped out.

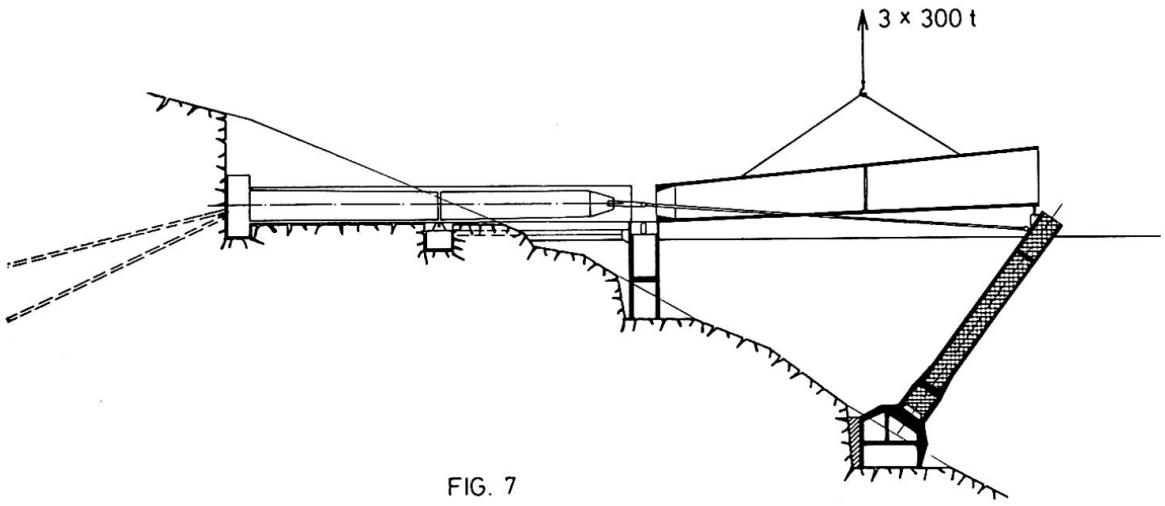

FIG. 7

After that the box structure,prefabricated on the shores in three segments (300 t each), has to be assembled above the arch oblique pier by the floating cranes (see Fig. 7.).Upon completion of assemblage, their transversal and longitudinal joints should be concreted too.

The supporting blocks are the last to be concreted, that is to say the concreting shall be performed above the pier,as well as the joint of the oblique pier and horizontal box structure at the place where the reinforced concrete arch is leaning (see Fig. 8.).

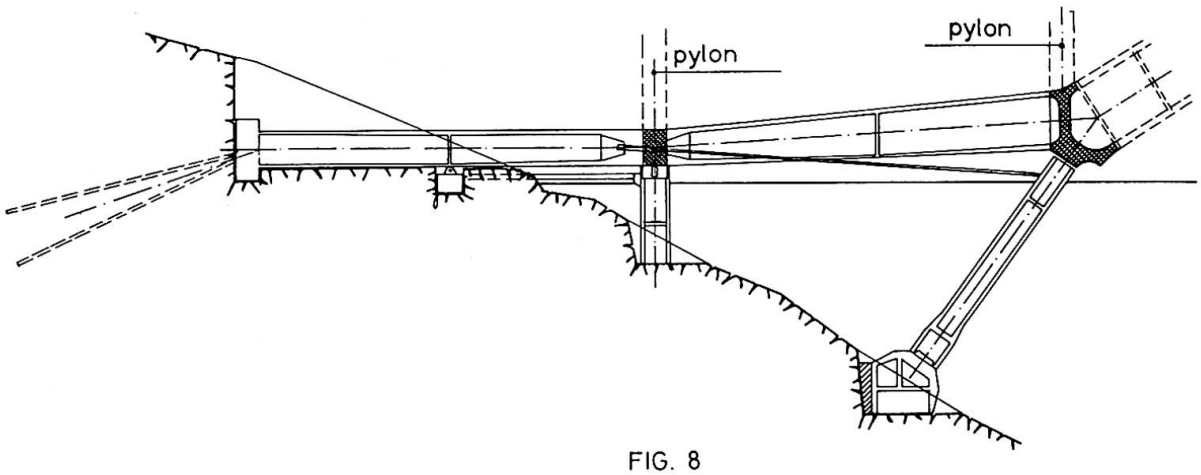

FIG. 8

Further,construction of the object should be continued upon estabilishment of this structure designed for the purpose of transfer of the arch reaction to the ground. Applying a sliding shuttering,the piers determined to sustain the superstructure in the final stage, are to be concreted as the first.In the arch construction they shall serve for suspension of the auxiliary steel ties, which, on the other hand, carry the suspendingly installed arch consisting of prefabricated reinforced concrete elements.

The installation of the arch has been predicted to begin in the early 1977.

SUMMARY

The foundation of a reinforced concrete arch bridge is described. The arch's reaction is transmitted to the ground through a horizontal box structure and an oblique pier, founded 19,00 m below the sea surface. The greatest part of this foundation structure will be composed of prefabricated reinforced concrete elements, which will be placed into the final position by a floating crane of great bearing capacity.

RESUME

Les fondations d'un pont en arc, en béton armé sont décrites. La réaction de l'arc est transmise au sol par l'entremise d'une structure en caisson horizontale et d'une pile oblique fondée à 19 m au dessous du niveau de la mer. La plus grande partie de la structure de fondation est composée d'éléments préfabriqués en béton armé, qui sont mis en place à l'aide d'une grue flottante de grande capacité.

ZUSAMMENFASSUNG

Die Fundation einer Stahlbeton-Bogenbrücke wird beschrieben. Die Bogenreaktion wird durch ein horizontales Kastentragwerk und einen Schrägpfeiler in den Baugrund geleitet. Die Fundamentkonstruktion liegt 19 m unter Meereshöhe. Der grösste Teil der Konstruktion besteht aus vorfabrizierten Stahlbetonelementen, welche mittels eines Schwimmkrans an Ort und Stelle versetzt werden.

Field Observation of Long Span Bridge Foundation Designed on the Results of Models Tests

Mesures in situ sur les fondations d'un pont de grande portée dimensionnées sur la base d'essais sur modèle

Messungen an Fundationen einer weitgespannten Brücke, welche auf Grund von Ergebnissen von Modellversuchen bemessen wurden

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1. Introduction

The main caissons of HAMANA Bridge was designed on the results of model tests, because the method of designing such a multi-cell caisson had not been established yet, as is stated in the Preliminary Report "Model Test for Design of Long Span Bridge Foundation" written by the authors. The caissons were sunken in March 1974 and the construction of superstructure was completed in August 1976. During the construction, the stress of one of these caissons was measured and compared with the results of model tests in order to confirm an actual state of the caisson designed in this way.

This paper shows the method of measurement and consideration of the result of measurement.

2. Instruments

Strain-gages and reinforcement-stress-gages, which are all Carlson type gages, were embedded in four sections, i.e. 12 strain-gages were embedded in the lower part of the pier to check the load from superstructure; 2 strain-gages and 6 reinforcement-stress-gages, in the top and the bottom of the slab to know its bending; 8 strain-gages, in the upper part of the bulkheads to know the transmission of load. In each section, a null-stress-gage was embedded in order to cancel the effects of creep or drying shrinkage in the measured data.

3. Loads under Consideration

The measurement was conducted for 21 months from the beginning of construction of the pier to the completion of the girder. The loads in this period consisted of the vertical load and the moment load which came through the pier. The vertical load was gradually increasing by the selfweight of pier and girder. The moment load, which was caused by the unbalanced weight of the main and side girders fluctuated according to the stage of construction of the girders.

The maximum values of these loads per one pier were 12,000ton and 27,000 ton-m respectively.

4. Results and Consideration

The measured strains were compared with calculated strains which were obtained by applying the strain-load relation based on the model tests to the

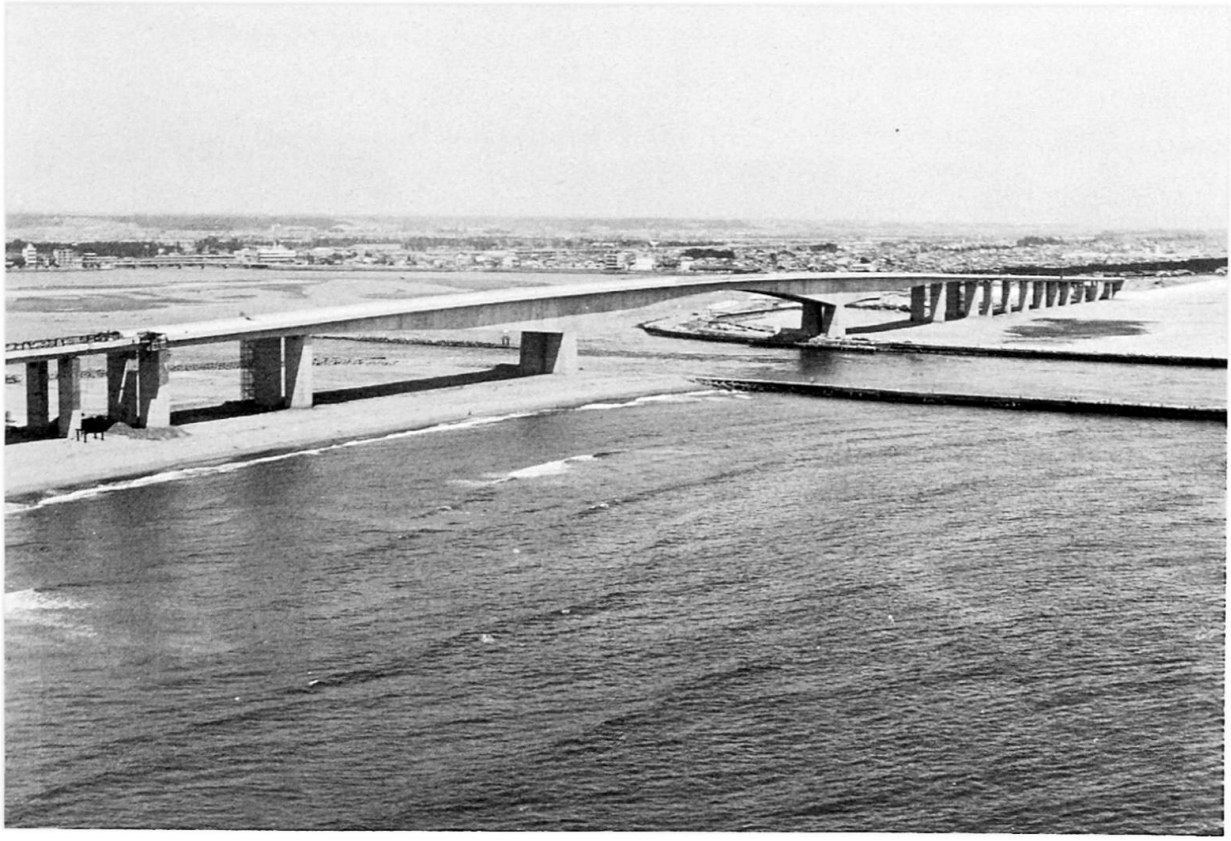

Photo. 1 Nearly Completed HAMANA Bridge

estimated loads depending on the stage of construction.

The strains detected at the pier showed good agreement with the calculated strains. This means that the slab of the casson was loaded by the just load as was estimated. The strains detected at the slab and the bulkheads also showed good agreement. This fact strongly supports the conception which was used in the design.

Considering the state of the slab, which was regarded as of great importance in the design, it can be considered that no tension cracks occured in the concrete during construction, because the maximum tension strain near the lower surface was $70\sim90\times10^{-6}$ and less than the general cracking strain of concrete. This also means that the slab will not crack under service loads, because the loads in these two conditions are almost equal.

Through the above study it has been certified that the calculated strains agree with the measured strains unless the slab cracks. But under the design load during earthquake, the slab will crack and its bending stiffness will reduce, therefore, the stress distribution is different from the results of model tests. In order to deal with this effect of cracking, Finite Element Method(F.E.M.) mentioned in the preliminary report had been used. Depending on the result, the reduction of bending stiffness of the slab resulted in slite increase of vertical stress in the bulkheads beneath the pier while the bending moment of the slab decreased. And in the actual design this increase of vertical stress had been estimated with sufficient safety.

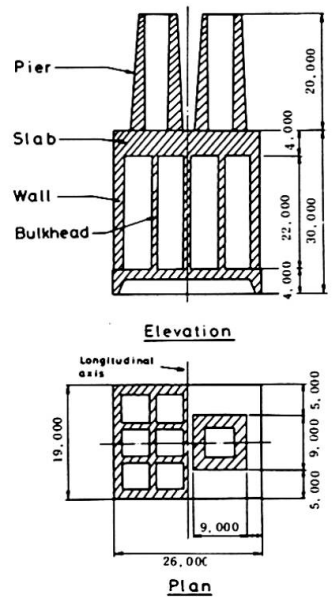

Fig.1 Caisson of HAMANA Bridge

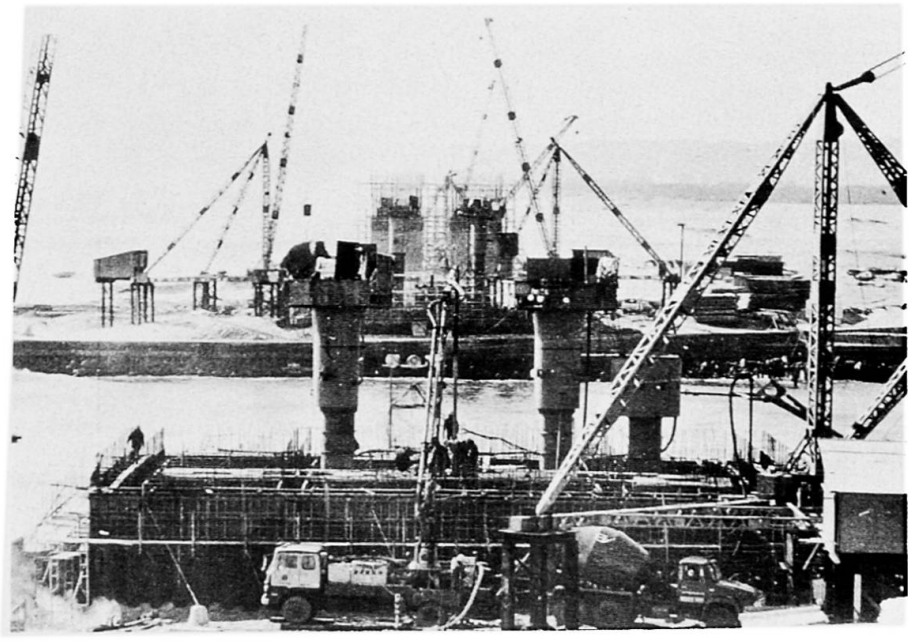

Photo. 2 Caisson under Construction

SUMMARY

After the multi-cell type huge caisson of HAMANA Bridge had been designed on the results of model tests as is stated in the Preliminary Report, the stress measurement during construction was carried out to know the actual state of these caissons. The results of measurement showed good agreement with the designed values and the validity of this design method was certified.

RESUME

Les caissons multicellulaires en béton armé des fondations du pont de HAMANA ont été dimensionnés sur la base d'essais sur modèle (voir Rapport Préliminaire). Les auteurs ont pratiqué une série de mesure de contraintes pendant la construction pour connaitre l'état réel du caisson. Les résultats montrent une bonne correspondance avec les valeurs calculées, ce qui prouve la validité de la méthode de calcul employée.

ZUSAMMENFASSUNG

Nachdem die vielzelligen Senkkästen der HAMANA Brücke auf Grund von Ergebnissen von Modellversuchen bemessen worden waren (siehe Vorbericht), wurden Baustellen-Messungen durchgeführt, um das tatsächliche Verhalten dieser Senkkästen zu untersuchen. Die Ergebnisse der Messungen zeigen gute Uebereinstimmung mit der Berechnung, wodurch die Gültigkeit dieser Bemessungsmethode nachgewiesen werden konnte.

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V

Emploi des aciers à haute résistance et à protection naturelle pour les structures hautes ou à grande portée

Anwendung hochfester Stähle, inklusive wetterfester Stähle, für hohe und weitgespannte Tragwerke

Application of high-strength Steels including weathering Steels to high-rise and long-span Structures

Va

Comportement sous charges en incluant les constructions hybrides

Tragverhalten, einschliesslich hybride Tragwerke

Structural Behaviour including Hybrid Construction

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Problèmes de conception

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Comments by the Author of the Introductory Report

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Bemerkungen des Verfassers des Einführungsberichtes

BEN KATO
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Structural Behavior Including Hybrid Construction

The state-of-the-art on the structural behavior of members and structures made of high-strength steels is outlined in this report according to sub-items listed in Table 1. Characteristics of the structural behavior of those made of high-strength steels are discussed in relation to those made of mild steel in order to provide the information by which a designer can decide whether a grade of high-strength steel should or should not be used in a particular part or in a particular structure in association with the imposed loading conditions.

1.MATERIAL	← BEEDLE et.al.
2.TENSION MEMBERS	
3.BEAMS & BEAM-COLUMNS -In Plane Behavior-	← FISHER et.al. (composite beam)
4.STABILITY	
4.1.COLUMN BUCKLING	
4.2.LOCAL BUCKLING	← FUKUCHI et.al.
4.3.LATERAL-TORSIONAL BUCKLING	← FUKUMOTO SUZUKI et.al.
4.4.ROTATION CAPACITY	← BEEDLE et.al.
5.FRAMES	
6.PLATE & BOX GIRDERS	← KUNIHIRO et.al. MAEDA et.al. YAMASAKI et.al.
7.FRACTURE & FATIGUE	

Table.1 Outline of the Theme and Contributors

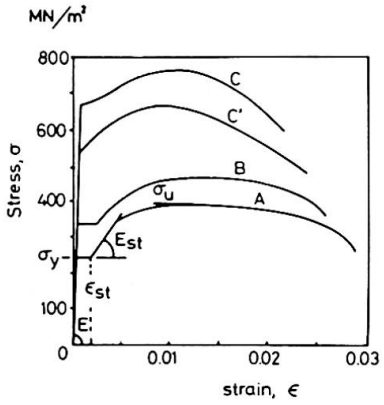
In this table,authors and topics which were contributed to the preliminary report are identified,and the synthesis of these contributions will be made in the following together with the key points of the introductory report.

Structural steels can be grouped into three classes as shown in Fig.1. Of which C-class steels are called as "extra high-strength steel",and mechanical properties of these steels in plastic region such as strain at initial strain hardening,strain hardening modulus and the yield ratio of material are considerably small compared with those of mild steel.

Dr.Beedle and his colleagues had studied a new type steel,designated as A572,Grade 65. This steel is a low-alloy columbium-vanadium steel and mechanical properties are in between of B and C-class steels as shown in Fig.2.

The poor capacities of deformation and energy absorbtion of tension members,beams, beam-columns and frames made of C-class steels are attributed to these characteristics of mechanical properties.

A-class: CARBON STEEL(MILD STEEL)
B-class: HIGH STRENGTH LOW-ALLOY STEEL
C-class: HEAT-TREATED CONSTRUCTIONAL ALLOY STEEL



CHARACTERISTIC VALUES IN PLASTIC RANGE
ϵ_{st} : Strain at Initial Strain Hardening
E_{st} : Strain Hardening Modulus
σ_y/σ_u : Yield Ratio of Materials

Fig.1 Classification of Structural Steels

The buckling behavior of members can be discussed by categorizing them into three groups according to their slenderness.
Since all compression elements have the same Eulerian strength regardless of the yield strength of the material,there will be no merit to use high-strength steels for slender members. For compression members with intermediate slenderness,the secondary factors such as

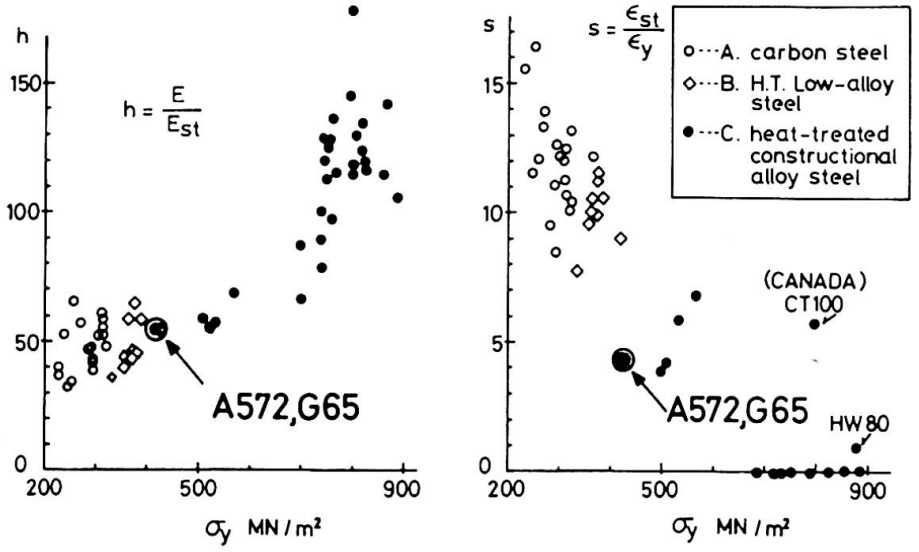


Fig.2 Characteristic Values in Plastic Region

residual stress,initial curvature and eccentricity have the greatest effect on tneir buckling strength. It is known that the effect of residual stress is less pronounced for higher strength steels than it is for mild steel,thus as the yield stress increases,initial curvature and eccentricity take on increasing importance in relation to residual stress. Dr.Fukumoto had carried out the lateral buckling tests of beams made of B and C-class steels with intermediate slenderness. A total 36 beam-type and 7 girder-type members was tested under uniform moment and moment gradient,and test results were compared with theoretical prediction. Fog.4 shows a non-dimensional presentation of the critical moment to slenderness relationship, and a comparison is made between the annealed beams and the as-weld beams having the same sizes for the residual stress effect. It has been concluded that the welding residual stress distributions may reduce the buckling strength for about 11% for B-class steel,and 6% for C-class steel against the beams without residual stress, and that initial lateral deflection of 0.1% of the beam length may explain the lower bound estimate against the plotted test results.

The deformability and ultimate strength of members or member elements with very small slenderness are the topics of increasing interest in relation to the development of plastic design and of earthquake resistant design. Because the behavior is determined almost entirely by the plastic properties of the material in such a short and stocky members,characteristic behavior of extra high-strength steels or C-class steels will be paramount in this range. Fig.5 shows the relationship between the rotation capacity and the slenderness of beams which subject to lateral buckling under uniform moment. Shown by the double circle is the Beedle's test

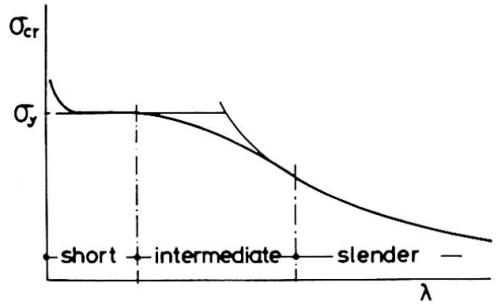


Fig.3 Categories of Slenderness of Compression Elements

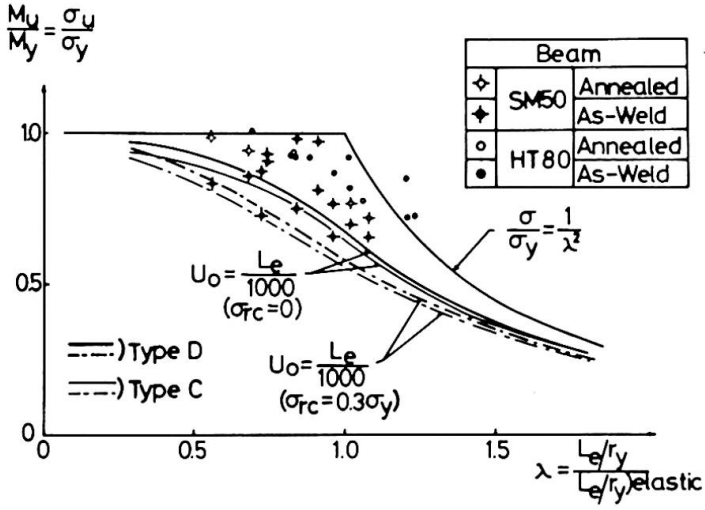


Fig.4 Test results and Theoretical Predictions

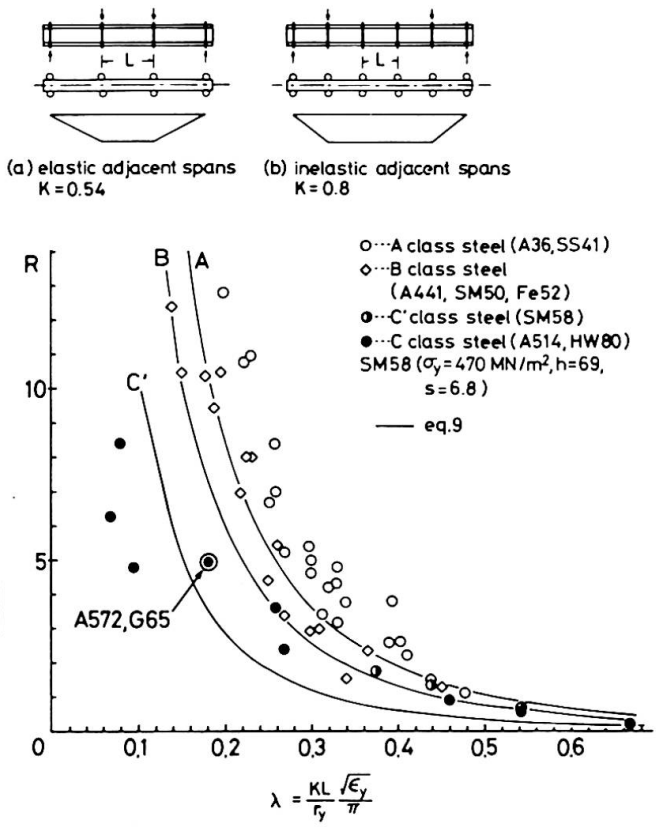


Fig.5 Rotation Capacity under Uniform Moment

result of which the beam is made of columbium-vanadium steel as refered previously. Rotation capacity falls in between those of B and C-class steels as is expected from mechanical properties of this steel.
Dr.Suzuli and Mr.Ono had carried out lateral buckling tests of beams and beam-columns with short slenderness. Steels used are of A,B and C-class. Fig.6 shows the relationship between the axial load and the slenderness of beam-columns. Theoretical prediction by Suzuki that assures the rotation capacity of 3 are shown by bold solid lines,and test results rotation of which were less than 3 are checked by x-mark. On the other hand,a suggested axial load limitation given by eq.(12) in the introductory report is shown by the fine solid lines. It seems that eq.(12) be not satisfactory for C-class steel. Considering the fact that the information on the rotation capacity of beam-columns is insufficient as yet,this contribution is very important. Test results on beams reported by Suzuki were already refered in the introductory report.

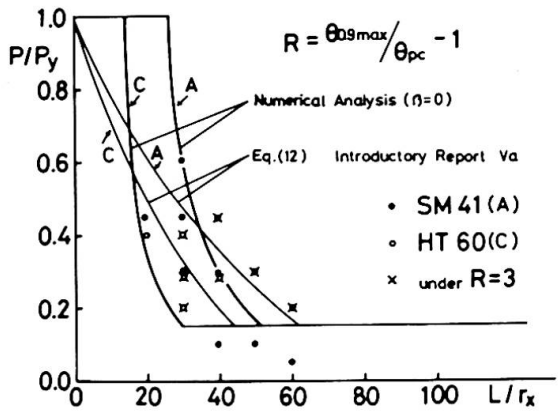


Fig.6 Rotation Capacity of Beam-Columns

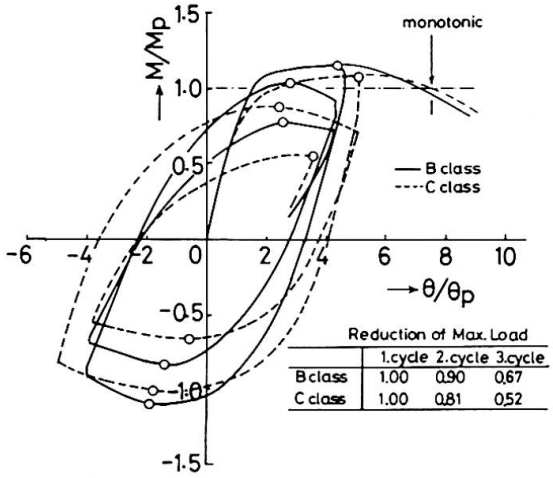
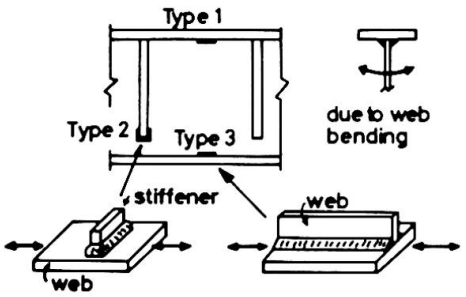


Fig.7 Cyclic Behavior of Beams

Behavior of beams under cyclic loading were investigated by Dr.Fukuchi and his colleagues. Fig.7 shows cyclic behavior of beams which subject to lateral and local buckling in their course of loading. Beams made of B-class and C-class steels which have almost the same rotation capacity when subjected to monotonic loading were tested and compared in this figure. Reduction of maximum load at each cycle is shown in the table,and it can be said that the rotation capacity of higher strength steel beams under cyclic loading will be more severely impaired than that expected from the monotonic test.

Hybrid beams are fabricated beams and girders which use a stronger steel in the flanges than in the web. The design of hybrid beams is based on the shakedown phenomenon, which ensures that the members will behave elastically after local yielding of the web. It seems that high-strength steels can be most efficiently utilized in this field of application as far as static loading is concerned. However this concept can not be applied to the fatigue design off hand,since metallurgical discontinuity and local stress condition around the welded joints will affect the fatigue strength considerably.

Typical patterns of fatigue cracks which will occur in hybrid girders are shown in Fig.8.



Fug.8 Typical Patterns of Fatigue Cracks

Type 1 crack will be caused by web breathing due to initial bending of the web, Type 2 crack initiated at the toe of vertical stiffener-to-web fillet weld can be compared to that of a transverse non-load carrying fillet welded joint,and Type 3 crack initiated at tension flange-to-web fillet weld can be compared to that of a longitudinal fillet welded joint as shown in the figure.

Three papers were contributed to this topic. Dr.Maeda and his colleagues had studied Type 1 and Type 2 fatigue phenomena and found out that Type 1 crack can be prevented up to 2 million cycles of loading by limiting out of plane movement of web plate, depending on web slenderness and rigidity of a horizontal stiffener, and also found out that Type 2 crack can be prevented by controlling a tensile web stress bellow the fatigue strength of transverse non-load carrying fillet welded joints.
Dr.Kunihiro's group and Dr.Yamasaki 's group had investigated the fatigue strength of Type 3.
Solid circles in Fig.9 represent the test results for hybrid specimens and open circles are for homogeneous specimens. As seen in this figure,test points are somewhat scattered and no statistical difference was found between fatigue strengthes of homogeneous and hybrid specimens. This might mean that the influence of geometrical and metallurgical imperfections of welded joints is large enough to cover up the characteristics of hybrid joints. In any case,the results of these two groups had supported the conclusion of Joint ASCE-AASHO Committee that "hybrid beams can generally be designed for fatigue as if they were made entirely of the grade of steel used in the flanges". In addition to these fatigue studies,Dr.Maeda and Dr.Kunihiro had discussed on the static behavior and economy of hybrid girders, and Dr.Yamasaki had analysed the problem of fatigue crack propagation by applying a theory of fracture mechanics.

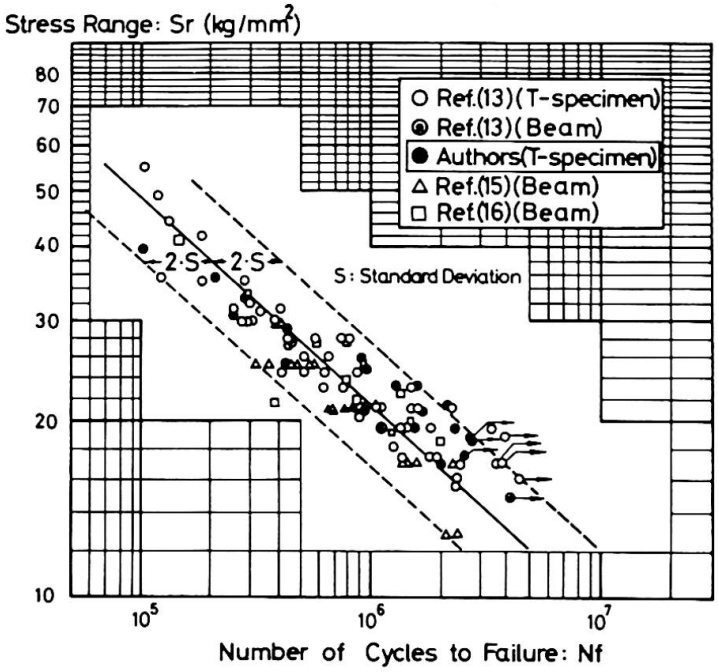


Fig.9 S-N Diagram of Hybrid Specimens

Finally,the design problem of composite beams were discussed by Dr.Fisher and his colleagues. With respect to the composite beam in which a concrete floor slab with formed steel deck and a steel beam made of mild steel are connected by means of shear connectors,a design formula based on the ultimate strength concept has been already proposed by Fisher himself. At present study,Dr.Fisher and his colleagues had examined the possibility of extensive application of high strength steel beams to this composite beam system by a series of tests,and concluded that "the flexural capacity of a composite beam utilizing high-strength steel is not adversely affected by the increased slab force and can be predicted provided that the connector capacity is known".

As reviewed herewith,the problems of fracture and fatigue in their proper sense were not contributed to the preliminary report,and the author hopes these problems would be discussed in prepared and free discussions.

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Bemerkungen des Verfassers des Einführungsberichtes

Comments by the Author of the Introductory Report

Remarques de l'auteur du rapport introductif

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Entwurfsprobleme

Die Berücksichtigung hochfester Stähle beim Entwurf von Ingenieurbauwerken kann nur dann erfolgreich sein, wenn deren besondere Eigenschaften im Hinblick auf das Tragverhalten sowie auf die speziellen Herstellungs- und Montageprobleme beachten werden.

Aus diesen gegenseitigen Rückwirkungen ergibt sich, daß die Probleme der drei Unterthemen ineinandergreifen und ihre Grenzen fließend sind, wie auch die Beiträge der verschiedenen Autoren gezeigt haben.

Das unterschiedliche elastisch-plastische Verhalten der normalen und der hochfesten Baustähle wird von AKYAMA vorteilhaft für den erdbebensicheren Entwurf von Hochhäusern herausgestellt. Das über die Plastizierungsgrenze der normalen Stähle hinauswirkende elastische Verhalten der hochfesten Stähle verteilt die Erdbebeneinwirkungen über die Höhe des Gebäudes und vermeidet dadurch Beanspruchungskonzentrationen in den einzelnen Stockwerken. Deshalb schlägt der Verfasser Hybridkonstruktionen vor (Vorbericht Seite 445) mit vertikalen Korsettstäben aus hochfesten Stahlgliedern in der Mitte oder an den Außenseiten des Hochhauses, damit die Schwingungsenergie durch eine kombinierte plastische Deformation der normalen und elastische Verformung der hochfesten Stahlglieder abgebaut werden kann. Wie Vergleichsuntersuchungen ergeben haben, genügt bereits ein Anteil von 10 % bis 15 % hochfester Stahlstützensysteme im Verbund, um die Energiekonzentration um 50 % zu vermindern. Die Berechnung solcher hybrider Tragwerke für den Entwurf erdbebensicherer Bauten nach AKYAMA zeigt gute Übereinstimmung mit Modellversuchen.

Daß hochfeste Stähle nicht nur im Verbund mit normalen Baustählen, sondern als Stützen auch im Verbund mit Unterzügen und Decken aus Stahlbeton vorteilhaft verwendet werden können, zeigt der Beitrag von GRAVERT. Während für den erdbebensicheren Entwurf von Hochhäusern die hochfesten Stahlglieder lastverteilend wirken, ermöglichen sie hier durch Lastkonzentration auf wenige geschweißte Stahlstützen mit Blechdicken bis zu 100 mm einen erheblichen Nutzflächengewinn (Vorbericht S. 451) gegenüber Stahlbetonstützen, die fast die vierfache Fläche benötigt hätten. Die geschoßweise horizontale Stabilisierung der Stahlstützen durch die Stahlbetondeckenscheiben läßt eine Bemessung auf Knicken nicht maßgebend werden. Die wegen ihrer größeren Lastaufnahmefähigkeit relativ größeren Stauchungsmaße der hochfesten Stahlstützen gegenüber den dehnsteiferen Stahlbetonkernen werden durch Auflagerüberhöhung der Geschoßdecken im Rohbauzustand weitgehend ausgeglichen. Der Beitrag von GRAVERT zeigt, daß der Einsatz von hochfesten Baustählen beim Entwurf von Ingenieurtragwerken nicht nur vom Standpunkt der kostengünstigeren Konstruktion, sondern auch von der wirtschaftlicheren Nutzung her zu beurteilen ist.

Das bekannte Problem der Mitwirkung der Längsträger bei der Hauptträgerbeanspruchung von Fachwerkeisenbahnbrücken mit offener Fahrbahn ist, wie von STRELETZKIJ erläutert wird, beim Einsatz hochfester Stähle für die Hauptträgergurte infolge deren relativ größeren Dehnungsnachgiebigkeit in noch stärkerem Maße zu beachten. Eine genaue computerisierte Berechnung unter Berücksichtigung des räumlichen Zusammenwirkens aller Tragwerksteile (Vorbericht S. 469) und besondere konstruktive Maßnahmen bei den Verbindungen senken dank der Verwendung hochfester Stähle für die Gurte den Stahlverbrauch für diese hybriden Eisenbahnbrücken um 5 % und verbessern darüber hinaus nach STRELETZKIJ auch die Betriebssicherheit und die sonstigen Nutzungseigenschaften. Insbesondere wird ein Freivorbau ohne Montageverstärkungen ermöglicht.

BOLSCHAKOW und POTAPKIN berichten über die Anwendung hochfester Baustähle in weitgespannten Straßen- und Eisenbahnbrücken. Die Festigkeitsuntersuchungen erfolgten durch die Autoren unter Berücksichtigung einer maximalen plastischen Restdehnung auf der Basis der Plastizitätstheorie. Für den Dauerfestigkeitsnachweis wurden Abminderungsfaktoren sowohl für den hochfesten Grundwerkstoff als auch für die hochfesten Verbindungen ermittelt. Die Abminderungsfaktoren für den Knicknachweis geschweißter und gewalzter Breitflanschprofile wurden unter Berücksichtigung von Eigenspannungen berechnet. Auch für längsversteifte Platten in Druckgurten von Kastenträgern im überkritischen Bereich und für kombinierte Verformungen von Trägern im elastisch-plastischen Bereich werden Beziehungen angegeben.

Um die Effizienz hochfester Stähle mit 50 und 60 kp/mm^2 Streckgrenze zu untersuchen, wurden 3 Stahlbrückentypen entworfen. Bei einer Fachwerkeisenbahnbrücke wurden mögliche Gewichtseinsparungen von 2 bis 6 %, bei einer Kastenträgerautobahnbrücke mit orthotroper Stahlfahrbahn von 5 bis 6 %, aber mit Stahlbetonfahrbahn von 20 % ermittelt. An ausgeführten Brücken (Vorbericht S. 463) wird berichtet über eine Autobahn-Bogenbrücke aus hochfestem Stahl mit 50 kp/mm^2 Streckgrenze, über eine vollgeschweißte Autobahnrahmenträgerbrücke aus hochfestem Baustahl mit einer Streckgrenze von 60 kp/mm^2 und eine Verbundbrücke mit zwei Hohlkästen, die ursprünglich aus Stahl mit Streckgrenze 40 kp/mm^2 ausgeführt werden sollte, für die das Stahlwerk aber hochfesten Stahl S 60 geliefert hat. Nach Auffassung von BOLSCHAKOW und POTAPKIN haben die hochfesten Stähle S 50 und S 60 bisher nicht die erwarteten Vorteile gebracht, insbesondere wegen der im Verhältnis zur Streckgrenzenerhöhung nicht mitwachsenden Dauerfestigkeit. Hervorgehoben wird ihre Widerstandsfähigkeit gegen niedrige Temperaturen.

Kriterien für die Dauerfestigkeitsnachweise der Honschu-Schikoku-Hängebrücken, die als kombinierte Autobahn-Eisenbahnbrücken mit hochfesten Baustählen der Festigkeitsklassen HT 60 bis HT 80 geplant sind, geben TAJIMA, OKUKAWA und TANAKA an. Versuche an Schweißverbindungen erlaubten die Festlegung von zulässigen Dauerfestigkeitsspannungen unter Berücksichtigung von 7 Kerbgruppen für Normal- und Schubspannungen (Vorbericht S. 479). Für jede dieser Kerbgruppen wurden Dauerfestigkeitsgrundwerte ermittelt. Zur Berücksichtigung des Mittelspannungseinflusses sind einfache Beziehungen in Abhängigkeit vom Spannungsverhältnis k angegeben, die unabhängig von der Kerbgruppe und für die Festigkeitsklassen 60 kp/mm^2 bis 80 kp/mm^2 gleich sind. Bei Schweißverbindungen aus hochfesten Stählen kann durch verschiedene Nachbehandlungsverfahren eine deutliche Erhöhung der Dauerfestigkeit bewirkt werden, wodurch eine Einstufung in die nächst höhere Kerbgruppe zulässig wird. Hervorzuheben ist, daß TAJIMA, OKUKAWA und TANAKA durch eine Schadensakkumulationsrechnung (cumulative damage) auf der Basis der Miner-Regel für die in 100 Jahren auftretenden Belastungswechsel Verbessungsfaktoren für die Dauerfestigkeitswerte ermitteln. Die bei der Miner-Rechnung zu Grunde gelegte normierte Wöhlerlinie (S-N-Curve) verläuft etwas flacher als die in Deutschland verwendete, d.h. bei gleicher Dauerfestigkeit wird die Zeitfestigkeit für die hochfesten Stähle in Japan etwas niedriger eingeschätzt. In gewisser Übereinstimmung mit den japanischen Untersuchungen ergaben auch deutsche Versuche, daß bei Bauteilen mit geringen Kerbfaktoren, Beanspruchungen mit hohen Mittellasten, Belastungen mit geringer Lastspielzahl und Betriebslastkollektive- mit geringem Völligkeitsgrad, die hochfesten Stähle

auch bei schwingender Beanspruchung den niedriger festen Stählen überlegen sind.

HAJDIN berichtet, daß hochfeste Stahldrähte als Paralleldrahtbündel auch unter der schwingenden Beanspruchung einer Schrägseil-Eisenbahnbrücke vorteilhaft eingesetzt werden können, insbesondere, wenn bei günstigem Lastkollektiv das Spannungsverhältnis $\varkappa$ (Vorbericht S. 475) angehoben werden kann. Eine Vergleichsuntersuchung ergab eine Gewichtsersparnis von 25 % zugunsten des Paralleldrahtbündels gegenüber dem verschlossenen Seil. HAJDIN befürwortet auf Grund von Schwingfestigkeitsversuchen eine Erhöhung der zulässigen Schwingbreiten um 25 % für das Paralleldrahtbündel, dessen Korrosionsbeständigkeit ein umhüllendes Polyäthylenrohr, in das Injektionsmörtel eingepreßt wird, gewährleistet.

In Ergänzung zu meinem Einführungsbericht kann ich noch mitteilen, daß in einer kürzlich am Institut für Statik und Stahlbau der Technischen Hochschule Darmstadt abgeschlossenen Forschungsarbeit [1] die Andwendbarkeit des vereinfachten Traglastverfahrens mit Fließgelenkhypothese für hochfeste Baustähle am Beispiel des Rahmenträgers überprüft wurde. Experimentelle Untersuchungen haben gezeigt, daß der Bereich Verfestigungsdehnung ε_{St} minus Fließdehnung ε_F und der Verfestigungsmodul E_{St} auch für hochfeste Stähle hinreichend groß sind, um in allen Fällen zu einer ausgeprägten Fließgelenkwirkung zu führen.

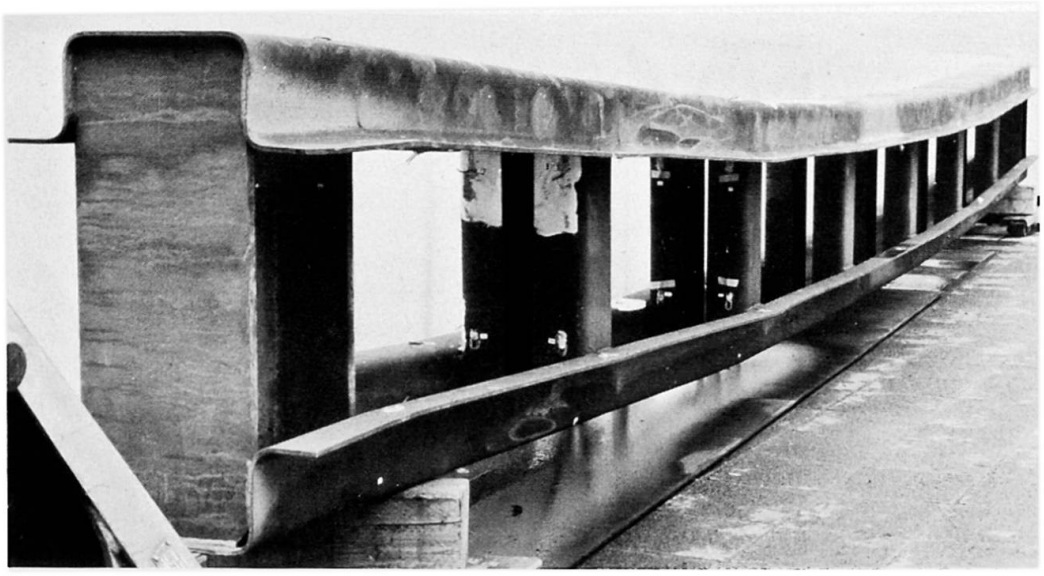

Bild 1

Im Versuch (Bild 1) geprüft wurden wirklichkeitsnahe Rahmenträger mit 8,40 m Spannweite aus abgekanteten Blechprofilen der 4 genormten Festigkeitsklassen mit Regelstreckgrenzen σ_F = 24, 36, 47 und 70 kp/mm^2 (Bild 2).

①	②	③	④	⑤
Stahl	Streckgrenze [kp/mm²]	$P_u / P_{gr,F}^{pl,N}$	$P_u / P_{gr,R}^{pl}$	$P_u / P_{gr,F}^{el}$
St 37	$\sigma_F = 24$	1,16	1,12	1,53
St 52	$\sigma_F = 36$	1,11	1,06	1,53
StE 47	$\sigma_F = 47$	1,05	1,01	1,40
StE 70	$\sigma_F = 70$	1,09	1,04	1,59

Vergleich der elastischen und plastischen Grenzlasten bei Rahmenträgern aus 4 Festigkeitsklassen.

Bild 2

Die Gegenüberstellung der im Versuch erreichten höchsten experimentellen Traglast P_u mit den theoretischen Grenzlasten, bei denen aber die konstruktiv bedingten Anschlußsteifigkeiten an den Knotenpunkten berücksichtigt wurden, ergibt folgendes: Für alle 4 Baustahlfestigkeitsklassen liegt die plastische Grenzlast $P_{gr,F}^{pl,N}$ mit Berücksichtigung des Normalkrafteinflusses und der tatsächlichen Streckgrenze σ_F mit 5 bis 16 % auf der sicheren Seite (Spalte 3). In unseren Untersuchungen hat sich ferner gezeigt, daß die gute Übereinstimmung mit den Versuchswerten immer noch auf der sicheren Seite lag, wenn man (Spalte 4) den rechentechnisch aufwendigen aber grenzlastmindernden Normalkrafteinfluß mit der gegenüber den tatsächlichen Lieferwerten statistisch niedriger liegenden normenmäßigen Regelstreckgrenze kompensiert. Eine Berechnung nach der elastischen Grenzlast bei unbeschränkter Gültigkeit des Hook'schen Gesetzes (nach Spalte 5) würde aber die Tragfähigkeit des Rahmenträgers um ca. 40 bis 60 % unterschätzen. Diese Versuche bestätigen, daß die plastische Grenzlastbemessung auch für hochfeste Baustähle anwendbar ist.

Meine Damen und Herren,
zusammenfassend läßt sich feststellen, daß beim Entwurf von Hoch- und Brückenbauten hochfeste Baustähle in vielen Fällen vorteilhaft berücksichtigt werden können, insbesondere bei Hybrid- und Verbundkonstruktionen. Wenn auch ihre Dauerfestigkeit wegen der gleichen Naturkerbigkeit nicht höher liegt als die der niedriger festen Stähle, so ist im Zeitfestigkeitsbereich und bei günstigen Lastkollektiven ihre höhere Festigkeit auch bei dynamisch beanspruchten Tragwerken ausnutzbar.

Insbesondere auf den Gebieten der Stabilität und der schwingenden Beanspruchung sind aber noch weitere Forschungsergebnisse erforderlich, um alle notwendigen Entwurfsparameter verfügbar zu haben.

[1] O. Jungbluth und H. Bahernejad:

" Zur Traglastbemessung bei Verwendung der hochfesten Baustähle StE 47 und StE 70 ",

Deutscher Ausschuß für Stahlbau, Forschung und Entwicklung im Stahlbau, 1977.

Comments by the Author of the Introductory Report

Remarques de l'auteur du rapport introductif

Bemerkungen des Verfassers des Einführungsberichtes

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Fabrication and Erection Problems

Session Vc is intended for discussions on fabrication and erection problems of long-span bridges for the application of high-strength steels. Since long-span bridge projects are going to be planned in the world, the subject will bring forward the most realistic and important problems for bridge engineers.

Although the introductory report on the present subject was not presented to our regret, we have three Preliminary Reports. Now, the author is going to discuss some problems together with the summary of the Preliminary Reports.

One of the Reports treated Saint-Nazaire Bridge in France which is the world-longest cable-stayed girder bridge with a center span of 404 m. The second report covered the choice of high-strength steels at Osaka Port Bridge in Japan, and at the third report the use of high-strength steels was discussed for support forces of steel bridge girders during their launching. It is thought that each of the reports is characteristics from the point of the present subject.

The author believes that it is the determination of an erection method to influence most on the safety and economy of long-span bridges. Four bridges accidents during their erection have been reported very recently, and the Merrison Committee Report was published and also at the Session VII, Professor Massonnet and Dr. Henderson will make some proposals on erection at their General Reports. It is advisable to consider fully such suggestions or recommendations at the design of future bridges.

Now, the author would like to emphasize that it might be dangerous to apply directly the engineering experience obtained from mild steels to the use of high-strength steels. Generally, uniform elongation of high-strngth steels will decrease with an increase of strength, and particularly, at quenched and tempered high-strength steels over 60 kg/mm^2, their yielding ratio will be increased and their stress-strain curve will be different from mild steels.

For example, judging from fatigue tests of steel plates with a notch, mild steels are a kind of stiff-type steel, but heat-treated high-strength steels become a kind of soft-type steel. From buckling, high-strength steels seem to be advantageous at the effect of residual stresses due to welding, but there are no sufficient test data to verify the effect. Therefore, careful considerations will be required at the design of details of a structure.

On the other hand, as welding problems, crack sensitivity due to welding will be higher at high-strength steels and particularly, brittleness of weld bond parts will be unavoidable in relation to weld heat input at heat-treated high-strength steels.

It will be concluded that a thorough study on the structural behavior of welded structures will be required and at the same time, material properties of high-strength steels and especially, weldability in relation to fracture mechanism have to be understood well at the further study.

Recenly, large-block erection methods have been applied to long-span bridges to satisfy environmental conditions, as seen in erection works of Saint-Nazaire Bridge and Osaka Port Bridge.

At the erection of Saint-Nazaire Bridge, the side spans were erected by a large-en bloc lifting-up method without any staging, and yet the main span was erected by a cantilever method. Since a uniform section of box gorders was used on account of easy works of fabrication and erection, working stresses induced during the erection forced the side spans to be made of steels more than 42 kg/mm^2 in yielding point. It is reported that such steels satisfied the required KCV-20°C in the direction of rolling and the perpendicular direction to it at the pre-and post welding.

At the erection and fabrication of Osaka Port Bridge, it is reported as follows:

1) The suspended truss span (span length is 186 m and steel weight including floor is approximately 4500 tons) was fabricated at a shop and towed to the site by a deck barge in 15,000 tons. Finally, it was lifted using a lifting equipment composed of wires and winches to the ends of the cantilever spans to the height of approximately 50 m above the sea level.

2) Out of the total steel weight 40,000 tons, 4197 tons and 1075 tons of high-strength steels of 80 kg/mm^2 and 70 kg/mm^2 grades, respectively, were used. Furthermore, the maximum plate thickness were 75 mm for the web plate of the chord and 100 mm for the bottom plate of the tower base.

3) To improve the weldability of members and elements, lamellar tear tests, restrained cracking tests, tests on the performance of corner weld joints, and tests to study on the various characteristics of the actual members using full scale models, were conducted prior to the fabrication.

It will be highly evaluated that with such well-prepared tests, the fabrication of the whole bridge structure was completed without any trouble.

It will be recognized at any rate that the use of high-strength steels at long-span bridges is somewhat advantageous because it can reduce their own weight at the erection. Fig. 1 illustrates the relation between allowable stress and cost ratio based on the cost of various steels in Japan. Although there are social and environmental requirements for bridges and all problems cannot be solved by one factor as shown in Fig. 1, it seems that the figure will give you a certain index to choose the material.

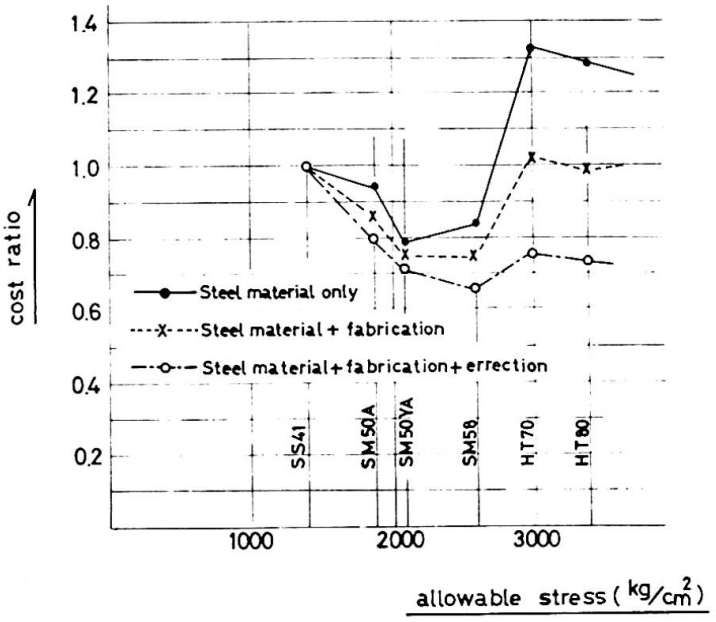

Fig. 1 Relation between allowable stress and cost ratio

At the paper presented by Prof. Bergfelt and Mr. Wilson, when a bridge was erected by launchimg, they observed experimentally girder behaviors due to local yielding, buckling and their combined action, and they concluded the use of high-strength steels would be advantageous to avoid damages of a structure due to concentrated loadings.

It is the author's opinion that the improvement of material properties in the direction of plate thickness is required from the point of stress transmission, when member sections of bridges are going to be larger and higher-strength steels are required for detailed design of welded sections. At 80 kg/mm^2 grade steels in Osaka Port Bridge, the improvement of manufacturing

steels was tried to reduce the impurity of steels, so that the contents of S and P were kept around or within 0.01 %. Then, HT80 steels could be satisfactorily applied to this bridge, as seen in Table 2, in comparison with 60 kg/mm^2 steels in Table 1.

Fig. 2 shows the relationship among sulphur content, elongation in the direction transverse to the plate, and non-metal inclusion. Fig. 3 gives the relationship between elongation in the direction transverse to the plate and crack-growth rate by Cranfield Test which is a representative test of lamellar tear due to welding.

Finally, the author suggests that, since it is very important to pay close attention to the improvement of material properties in the direction of plate thickness at the use of high-strength steels, fabrication and erection of long-span bridges have to be done in cooperation with manufacturers of steels.

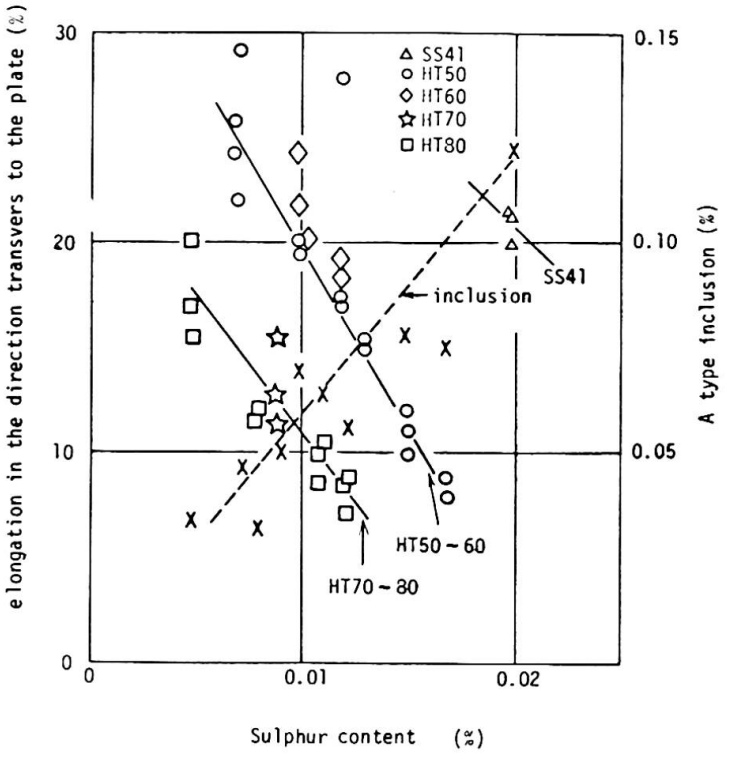

Fig. 2 Relationship among sulphur content, elongation in the direction transverse to the plate and non-metal inclusion

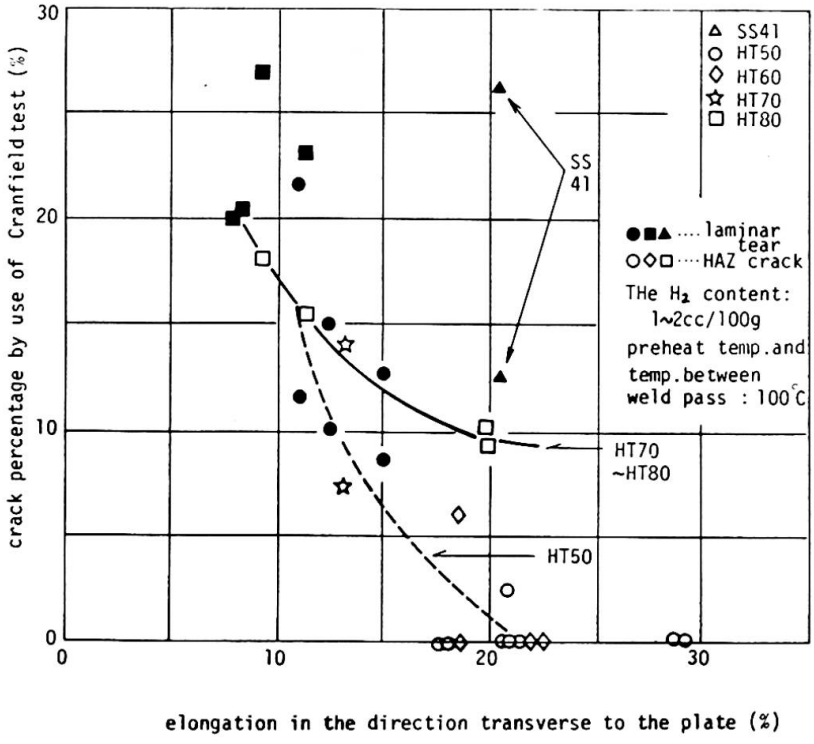

Fig. 3 Relationship between elongation in the direction transverse to the plate and Cranfield crack-growth rate

Table 1 Properties of 60 kg/mm^2 high-tensile steel plate

T*	Chemical composition											Properties in longitudinal and cross direction (JIS 4)								Properties in direction of thickness						Remarks
mm	C	Si	Mn	P	S	Cu	Cr	Ni	Mo	V	Ceq	P*	D*	Y.P.	T.S.	El.	R.A.	vEo	vTs	vTe	Y.P.	T.S.	El.	R.A.	vEo	vTs	
75	13	26	132	10	6	20	15	38	14	4	438	t/4	L	57.1	67.4	27.3	75.7	29.3	-63	-66	55.9	65.6	21.9	-	3.4	+15	D=14mm
													C	56.2	67.2	25.3	69.0	19.8	-42	-42							
85	15	30	137	18	10	3	3	2	24	6	461		L	59.2	68.7	21.8	-	VE-5 23.4	-30	-34	56.5	65.9	20.7	-	VE-5 1.5	+20	GL=50mm
													C	59.6	68.9	19.5	-	VE-5 6.6	+5	- 8							

Table 2 Properties of 80 kg/mm^2 high-tensile steel plate

T*	Chemical composition												Properties in longitudinal and cross direction (JIS 4)								Properties in direction of thickness						Remarks
mm	C	Si	Mn	P	S	Cu	Cr	Ni	Mo	V	B	Ceq	P*	D*	Y.P.	T.S.	El.	R.A.	vEo	vTs	vTe	Y.P.	T.S.	El.	R.A.	vEo	vTs	
33	12	27	87	11	7	27	47	99	45	3	15	500	t/4	L	80.4	85.0	25.6	69	19.4	-110	-99	76.6	82.3	15.0	33	-	-	
														C	81.7	85.3	21.1	55	6.4	-81	-92							
44.5	12	28	95	10	8	27	48	106	47	3	13	531		L	82.8	87.3	22.7	67	19.0	-91	-91	77.4	82.6	15.9	35	-	-	D=8.5mm GL=30mm
														C	83.7	87.8	20.8	58	6.8	-80	-97							
50	11	27	100	10	9	26	49	108	47	4	12	533		L	77.3	83.6	24.7	68	17.0	-50	-59	71.6	79.4	17.8	52	-	-	
														C	73.7	80.3	19.7	64	11.0	-60	-76							
50	10	31	90	7	5	26	47	103	47	3	12	502		L	81.7	86.0	23.6	72	20.9	-93	-92	79.8	83.1	11.5	33	7.2	-66	D=14mm GL=50mm
														C	81.6	85.8	22.6	66	132	-72	-66							
63.5	12	27	87	11	7	27	47	99	45	3	15	500		L	81.9	86.9	23.3	66	19.0	-90	-90	77.9	83.2	16.5	39	-	-	D=8.5mm GL=30mm
														C	80.5	85.4	23.6	60	9.2	-74	-77							
78	13	28	86	8	6	23	47	129	47	4	13	530		L	80.7	85.9	23.6	67	20.8	-96	-93	79.8	84.4	17.8	39	21.2	-122	D=14mm
														C	80.1	84.8	23.1	62	13.8	-76	-71							
100	12	27	89	8	5	24	52	143	48	3	18	542		L	77.1	84.2	25.2	71	23.4	-105	-93	74.8	81.5	22.0	-	11.9	-32	GL=50mm
														C	77.0	83.5	24.4	68	19.4	-95	-90							

C, Si, Mn, Cu, Cr, Ni, Mo, V : (10^{-2})
P, S, Ceq. : (10^{-3})
B : (10^{-4})

Y.P., T.S. : (kg/mm^2) , vTs, vTc : (C)
El., R.A. : (%)
vEo : (kg-m)

T*=thickness
P*=position
D*=direction of test

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Versuchsergebnisse vorgespannter Stahlträger im elastischen und elastoplastischen Zustand

Results of Tests of Prestressed Steel Beams in Elastic and Elastoplastic State

Résultats d'essais sur des poutres en acier précontraintes, à l'état élastique et élastoplastique

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Ziel der Prüfungen sechs vorgespannter Walzträger war die Ermittlung der wirklichen Wirkung dieser Träger, im elastischen wie auch im elastoplastischen Zustand und der Vergleich der Ergebnisse mit der Theorie /1/. Es waren grundsätzlich diese Gründe für die Wahl der Walzträger: Überprüfung der Theorie an herstellungsmässig einfachen Elementen, die geringe Anschaffungskosten haben, und Überprüfung der Verstärkungsmöglichkeit so wie auch der Tragfähigkeitserhöhung der Walzträger durch vorgespannte hochfeste Zugbänder.

Verwendet wurden Walzträger I PE 30 aus Stahl 11 373 mit 3800 mm Spannweite. Auf der ganzen Spannweitelänge waren die Träger mit einem hochfestem Zugband aus patentierten Drähten 20 Ø P 4,5 mm vorgespannt; das Zugband lief unter dem Trägeruntergurtflansch, parallel mit diesem, Bild 1. Die Träger N 1 bis N 6

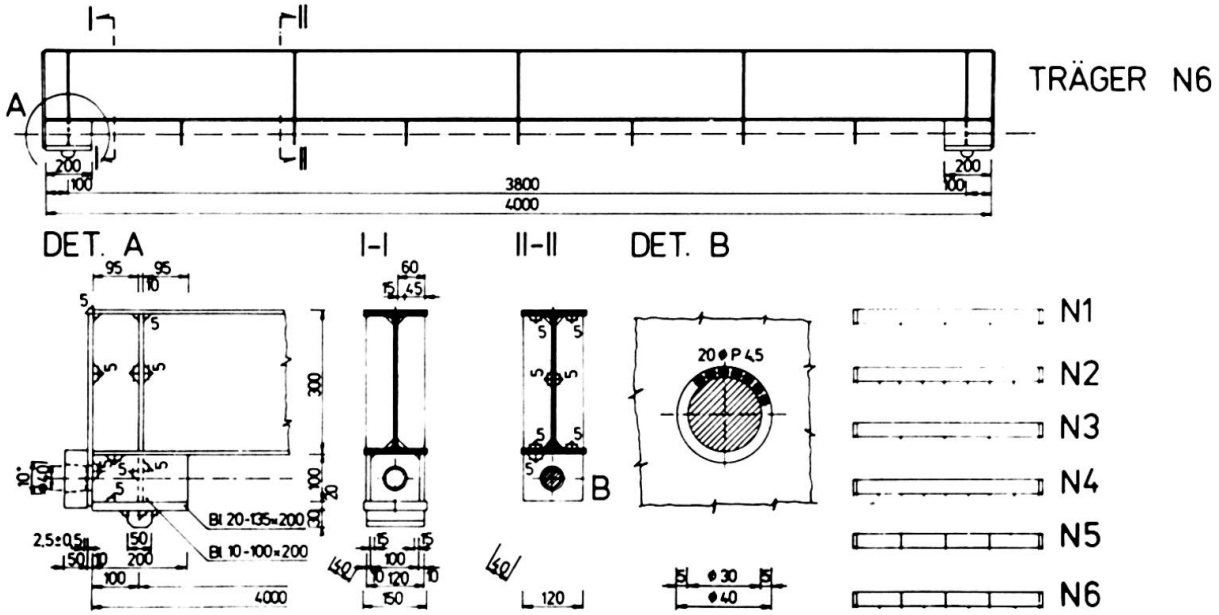


Bild 1. Prüfträger N 1 bis N 6

hatten unterschiedliche Zahlen der Stegdiafragmen, stabilisierenden Diafragmen unter dem Trägeruntergurtflansch und Einlagen im Zugbandinneren.

Die Ansicht der Gesamtanordnung der Prüfung ist am Bild 2 zu sehen. Oben sieht man den stützenden Hilfsträger, unten ist der Prüfträger, zwischen beiden Trägern sind drei Belastungspressen. Die Vorrichtung am Bild 2 rechts sichert die Stabilität der geprüften Konstruktion. Am Bild 3 ist das Zugband vom Ringquerschnitt an der Stelle des stabilisierenden Diafragma mit /Bild 3a/ und ohne /Bild 3b/ der pressenden Einlagen. Die Vorspannpistole /Bild 4/ stützte sich auf den ringförmigen Dynamometer, mit dem die Zugbandkraft kontrolliert wurde.

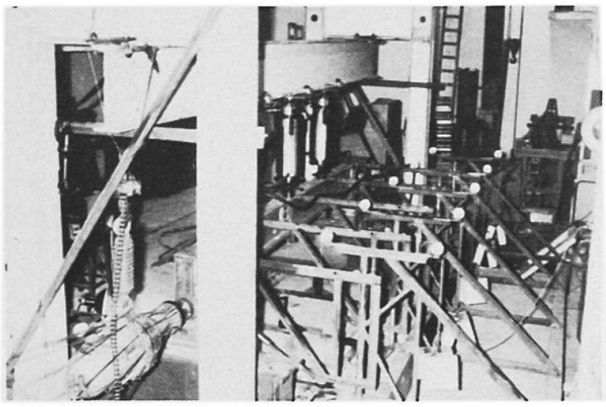
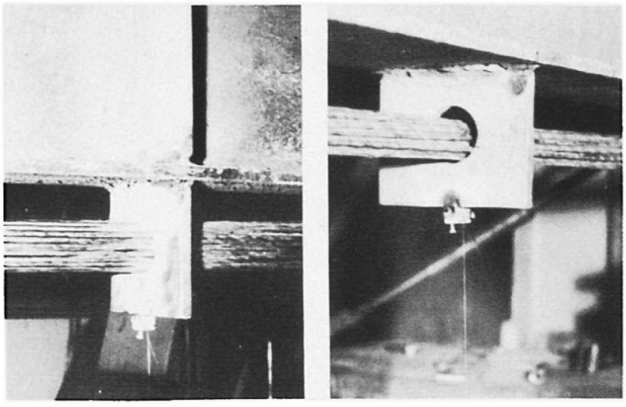


Bild 2.



a/ b/

Bild 3.

Die Spannungen im Träger wurden mit 16 Widerstandstensometern gemessen, die 10 cm von der Spannweitemitte des Trägers angeordnet waren. Die Durchbiegungen wurden mittels Durchbiegungsmessern an den Viertelpunkten der Spannweite /unter den Lasten/ gemessen, Bild 5.

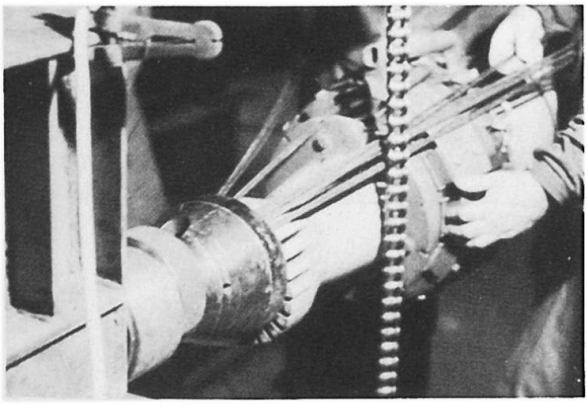


Bild 4.

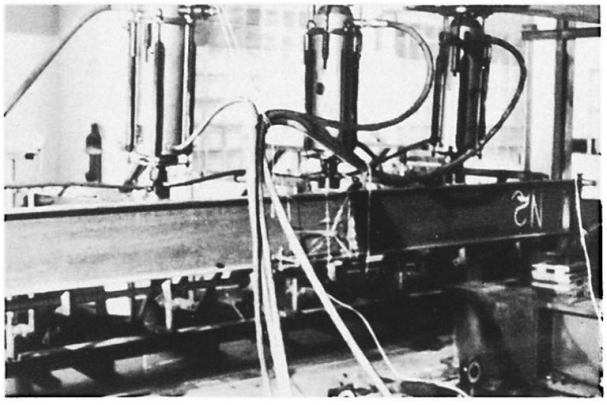


Bild 5.

Kontrolliert wurde auch die Stützensenkung. Bei zwei Trägern wurde die Trägerstabilität mit einer Ergänzungsprüfung gemessen, und zwar bei der Belastung nur durch die Vorspannkraft V: in einem Falle nach Beseitigung aller stabilisierenden Diafragmen /Bild 6/, in zweitem Fall beim Belassen dieser in der Spannweitemitte des Trägers.

Bei den Trägerprüfungen kamen folgende Belastungsstufen zur Geltung: Vorspannkraft V = 0-50-100-150-250-300 kN; lotrechte Lasten P = 0-20-5-40-5-60-5-80-5-90-5-100-110 kN, usw. bis zum Bruch der Konstruktion.

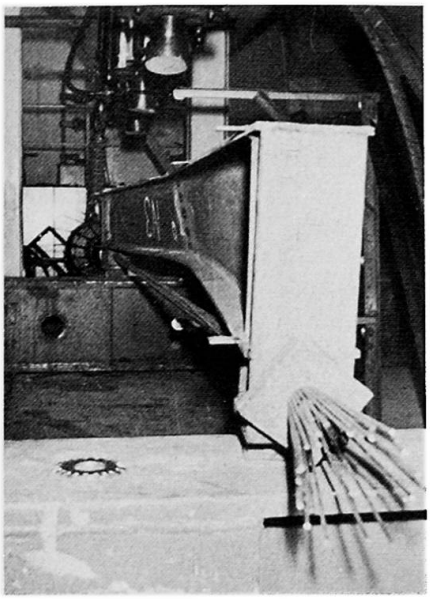


Bild 6. Stabilitätsverlust des Trägers, bei dem die stabilisierenden Diafragmen beseitigt wurden

Die Messergebnisse sind zu Diagrammen verarbeitet, von welchen je eines die Bilder 7 und 8 zeigen. Die erste Diagrammgruppe, die im Bild 7 gezeigt wird, stellt den Verlauf der Normalspannungen in der Trägermitte dar. Der Diagrammtyp im Bild 8 stellt das Anwachsen der verhältnismässigen Verformungen dar /lotrecht aufgetragen/, nachdem sie durch den Elastizitätsmodul multipliziert wurden /im elastischen Bereich also der Normalspannungen σ /in Abhängigkeit vom Anwachsen der Vorspannkraft V /horizontal nach rechts/ und der Belastung P /zurück in der Linksrichtung/. Ähnlich wurden die Messergebnisse der Durchbiegung y verarbeitet, Bild 9.

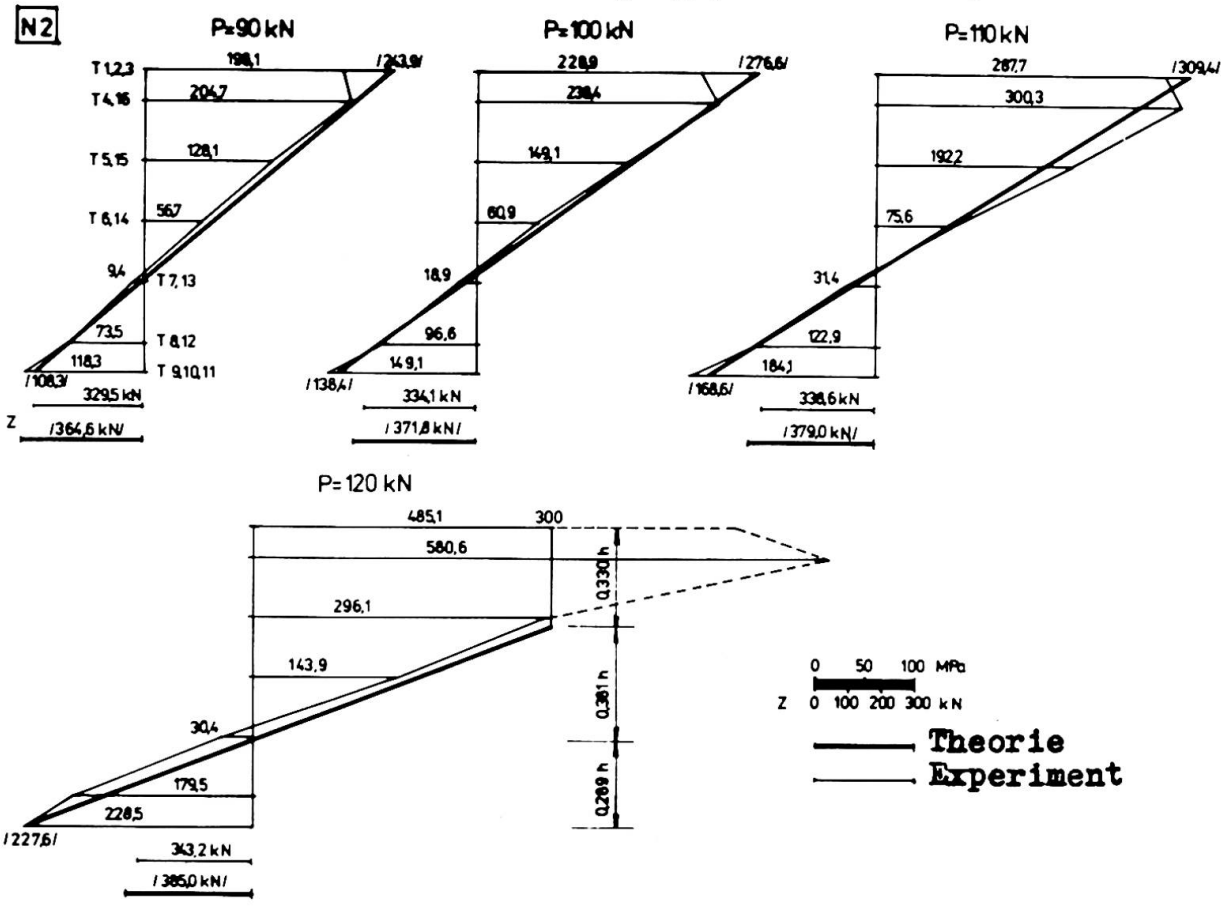


Bild 7. Träger N 2: Normalspannungen in der Mitte der Spannweite bei verschiedener Belastungsgrösse 3xP

Die Prüfungen brachten folgende Erkenntnisse:

Die erwartete Tragfähigkeit vorgespannter Träger im elastischen Zustand /bei der Normfliessgrenze σ_{fl} = 240 MPa grösser als Berechnungsbeanspruchung R = 210 MPa/ war 3x90 kN. Durch den Einfluss noch höherer wirklicher Fliessgrenze $\sigma_{fl} \doteq$ 300 MPa, stieg die theoretische elastische Tragfähigkeit auf cca 3x110 kN. Nach der Norm /2/ berech-

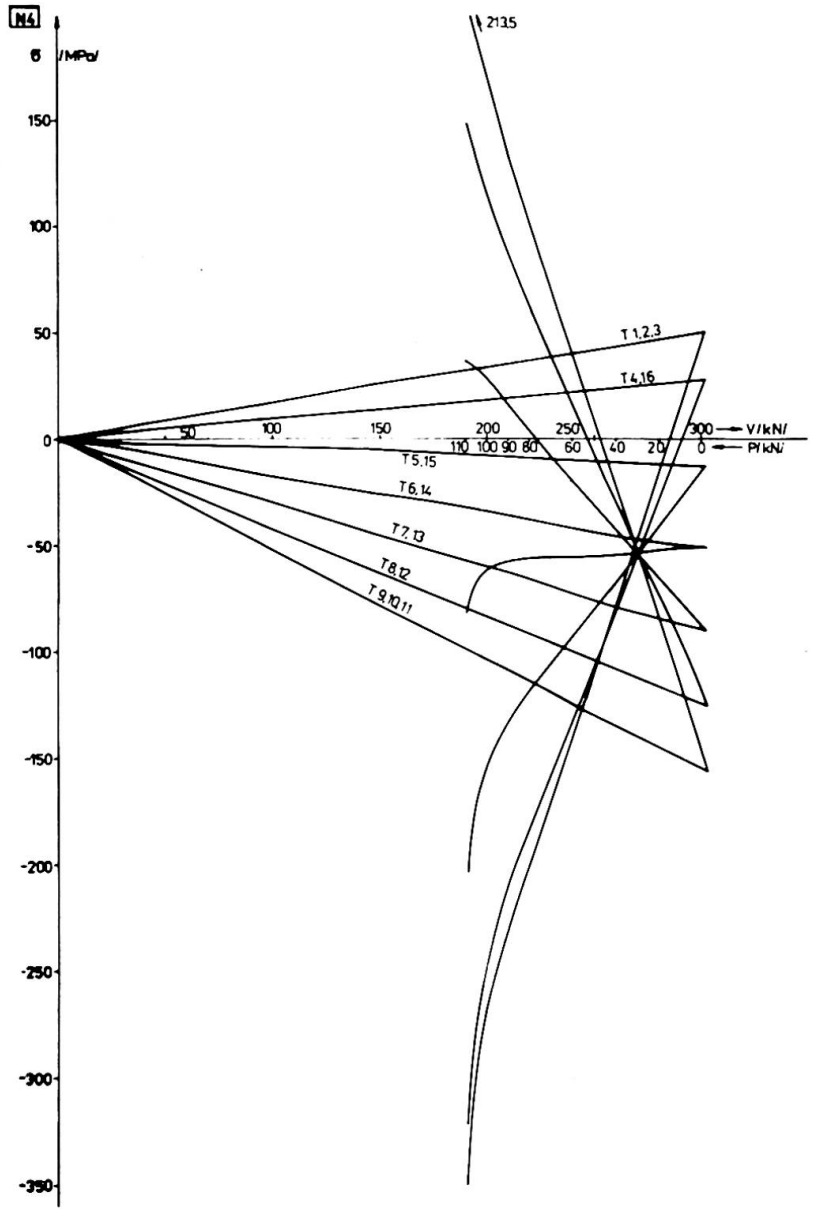


Bild 8. Träger N 4: Spannungsverlauf in Abhängigkeit von der Vorspannung V und Belastung P

net sich bei R = 210 MPa die tragende Last eines nicht vorgespannten Trägers I PE 30 mit 3x61,6 kN. In den ersten vier Trägern wurde eine hohe Tragfähigkeit erreicht, cca die doppelte des nach /2/ berechneten nicht vorgespannten Trägers. Bei den letzten zwei Trägern kam es zur Verringerung der Tragfähigkeit infolge Kippens; die Stützvorrichtung konnte den Obergurt nicht halten - auch so erreichte man eine um die Hälfte grössere Tragfähigkeit als beim nicht vorgespannten, nach /2/ berechneten, Träger.

Tensometrische Messungen zeigten eine sehr gute bis gute Übereinstimmung der Theorie und der Experimente. Es zeigte sich, dass mit den Vorkehrungen, die die Stabilität der Trägerdruckteile sichern, plastische Bereiche hervorgerufen werden können.

Die Durchbiegungsmessungen brachten folgende Erkenntnisse:

- Solange nur die Vorspannkraft wirkte, bestand eine sehr gute Übereinstimmung der theoretischen und gemessenen Durchbiegungen.

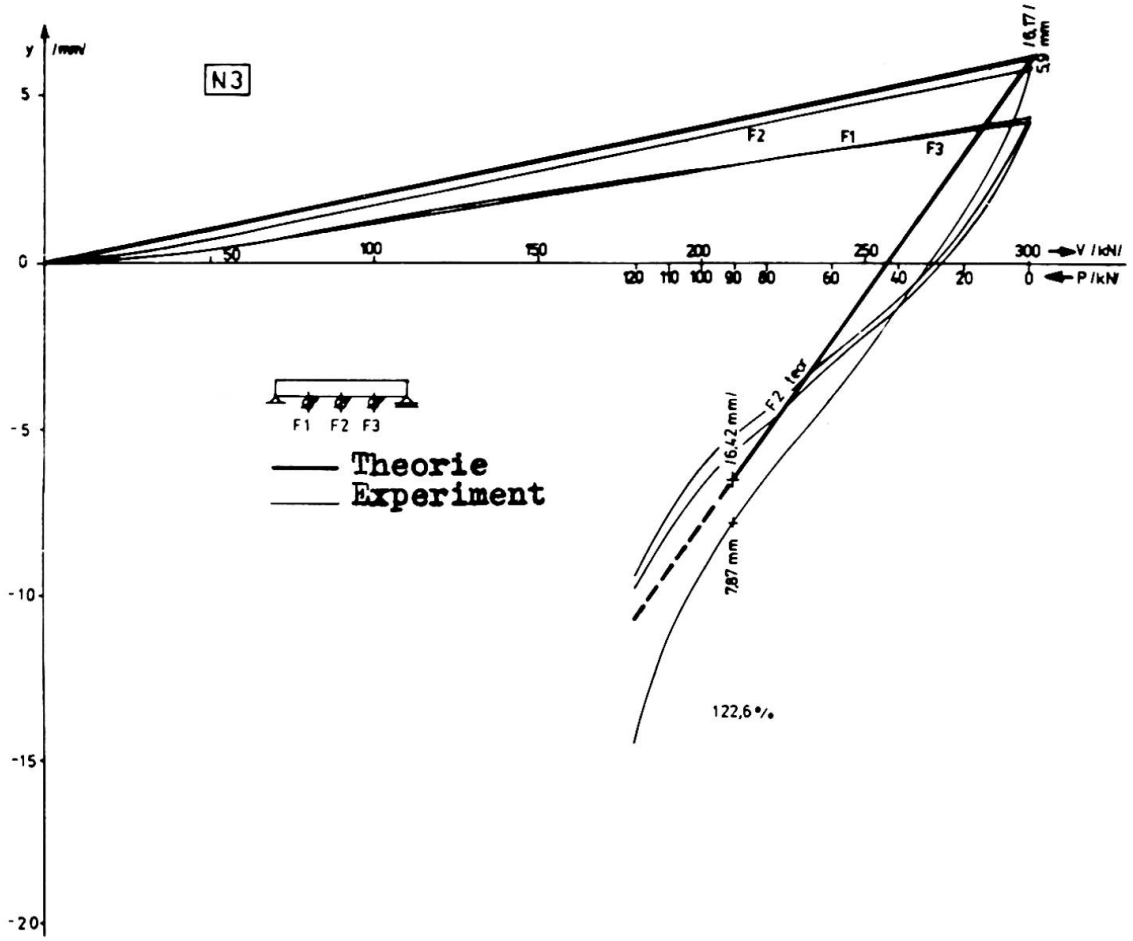


Bild 9. Durchbiegungen y des Trägers N 3 in Abhängigkeit von der Vorspannung V und Belastung P

In den Diagrammen, von denen eines am Bild 9 gezeigt wird, sind numerisch Kontrollwerte bei der Vorspannkraft V = 300 kN angegeben. Die theoretische Durchbiegung ist 6,17 mm /100%/; der durchschnittliche Wert aus den gemessenen Durchbiegungen ist 5,71 mm /92,5%/, also nur wenig geringer als berechnet. In den meisten Fällen wuchs die Durchbiegung linear.

- Wenn ausser der Vorspannung noch eine vertikale Belastung wirkte, waren die maximalen Durchbiegungen im Durchschnitt um 37% grösser als berechnet. Die Durchbiegungen waren grundsätzlich in guter Übereinstimmung mit den Spannungen, die gleichfalls höher waren als die berechneten.

Die Verschiedenheit der konstruktiven Lösung hat sich auf die Trägertragfähigkeit nicht ausgewirkt: Der Walzträger I PE hat einen genügend dicken Steg, so dass er hohe Einzellasten überträgt ohne dass der nicht ausgesteifte Steg lokal ausbeulte /I PE 30 überträgt verlässlich die Lasten 3x126 kN/. Der starke Untergurt braucht keine dichte Anordung der stabilisierenden Diafragmen. Auch ein Träger ohne stabilisierende Diafragmen übertrug eine Vorspannkraft von 400 kN /also um 100 kN mehr als der verlangte Wert/.

Die Prüfungen haben bestätigt, dass durch Vorspannung mittels eines hochfesten Zugbandes die Tragfähigkeit der Walzträger bedeutend erhöht werden kann, besonders im elastischen Bereich. Die Tragfähigkeit wächst weiter, wenn die plastische Reserve des Stahles

genützt wird. Die Ausnutzung der plastischen Reserve stösst auf Probleme der Stabilität. Infolge der Vorspannung, Nutzung der plastischen Reserve des Materials und bedeutend hoher Fliessgrenze des verwendeten Stahls erreichte man in 2/3 der Fälle der angeführten Experimente eine doppelte Tragfähigkeit gegenüber der, die die Berechnung des nicht vorgespannten Trägers /2/ ergibt.

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Behaviour of Hybrid Beam-Columns under Cyclic Loading

Comportement de montants hybrides soumis à des flexions cycliques

Verhalten von hybriden Stahlstützen unter zyklischer Biegebeanspruchung

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1. INTRODUCTION

The cyclic bending deformation behavior of the wide flange section was discussed by the senior auther(1),(2), and it was clarified that the cyclic bending moment-curvature relationship under the constant axial force asymptotes to the relationship of the pure bending due to the strain hardening effect. The cyclic bending behavior of the hybrid member is discussed here.

2.ANALYSIS

2.1. Analytical Model

A wide flange section is simplified into a three points model(2) such as shown in Fig.1. The area of the web is k times the one of the flange. The yield stress σ_{yw} of the web is ρ times the yield stress σ_{yf} of the flange. The stress-strain relationship of the material is tri-linear type such as shown in Fig.2. In general, with the increase of the yield stress σ_y, the yield ratio $\zeta=\sigma_y/\sigma_{max}$ increases and the strain hardening coefficient $\mu=E_{st}/E$ decreases. Fig.3 shows the relationship between the yield ratio ζ or the strain hardening coefficient μ and the yield stress σ_y. Indicating the stresses of each point as $\bar{\sigma}_i=\sigma_i/\sigma_{iy}(i=1,2,3)$, the axial force $N=nN_y$ and the bending moment $M=mM_y$ are obtained as follows:

$$n=\frac{\bar{\sigma}_1+\bar{\sigma}_3+\rho k\bar{\sigma}_2}{2+\rho k}$$

$$m=\frac{\bar{\sigma}_1-\bar{\sigma}_3}{2}$$

The stresses of each point are obtained by summation of the elastic stress and the residual stress such as shown in Fig.4, and are represented as follows:

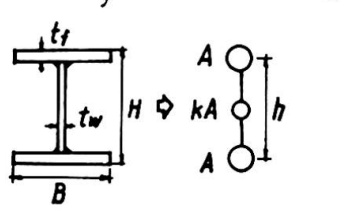


Fig.1 Three Points Model

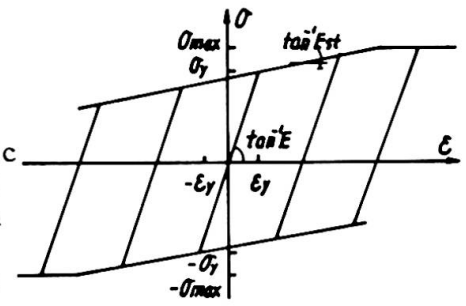


Fig.2 σ - ε Relationship

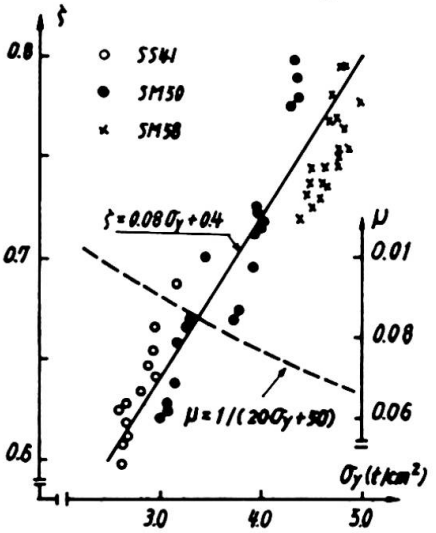


Fig.3 Yield Ratio ζ and Strain Hard. Coeff. μ

$$\bar{\sigma}_1=(2+\rho k)/(2+k)n+m+\xi$$

$$\bar{\sigma}_2=(2+\rho k)/\rho(2+k)n-2\xi/\rho k$$

$$\bar{\sigma}_3=(2+\rho k)/(2+k)n-m+\xi$$

where ξ is a parameter indicating the magnitude of the residual stress. The yield conditions of each point are as follows:

$$\alpha_i-2\leq\bar{\sigma}_i\leq\alpha_i$$

$$-1/\zeta_i\leq\bar{\sigma}_i\leq 1/\zeta_i$$

where α_i shows the subsequent yield stress. Fig.5 shows the yield polygon. The broken line shows the envelope of the yield polygon and a stress point can not go out of this range.

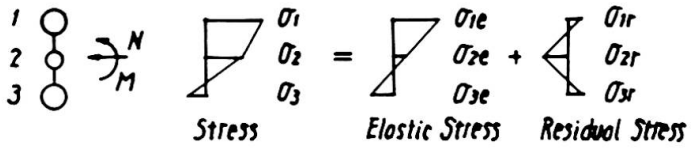


Fig.4 Stress Distribution

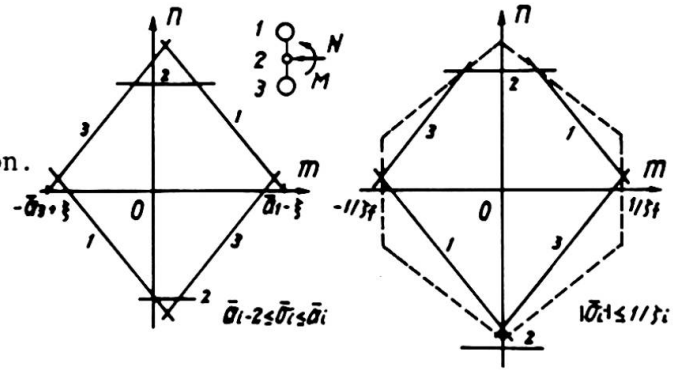


Fig.5 Yield Conditions

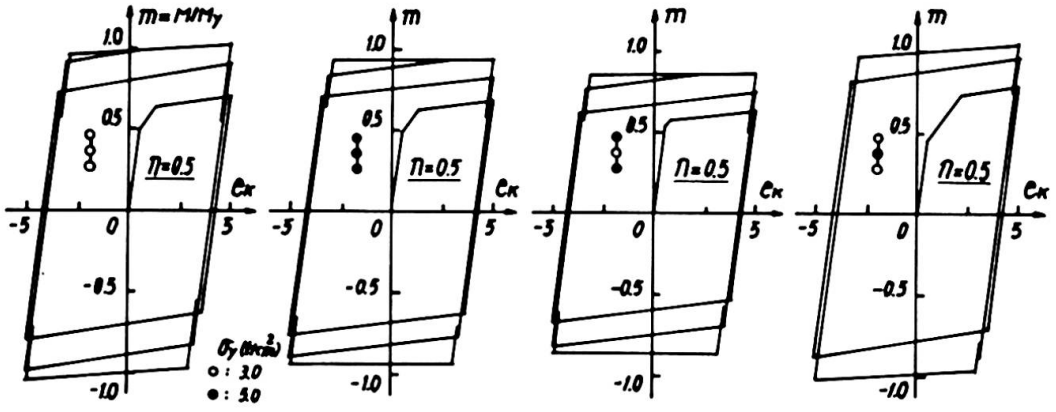
2.2. Bending Moment-Curvature Relationship

Fig.6 shows the bending moment-curvature relationship under the constant axial force of $N=0.5N_y$. Four types of the cross section computed are shown in Table 1. Figs.(a) and (b) correspond to the homogeneous members using a mild steel and a high strength steel, and Figs.(c) and (d) to the hybrid member using a high strength steel as the flange and the web, respectively. In each case, the relationships converge to the steady state loop at the three or four cycles. In the case of type A and D, the relationships converge to those of the pure bending due to the strain hardening effect, whereas in the case of type B and C, the loops converge to the steady state ones, the maximum bending capacity of which is somewhat smaller than that of the former case, due to the fully plastic state of the web and the compression flange.

Table 1 Types of Members

	Type	σ_{yf}(t/cm^2)	σ_{yw}(t/cm^2)
Homogeneous Member	A	3.0	3.0
	B	5.0	5.0
Hybrid Member	C	5.0	3.0
	D	3.0	5.0

σ_y(t/cm^2)	$\mu=E_{st}/E$	$\zeta=\sigma_y/\sigma_{max}$
3.0	1/110	0.64
5.0	1/150	0.80

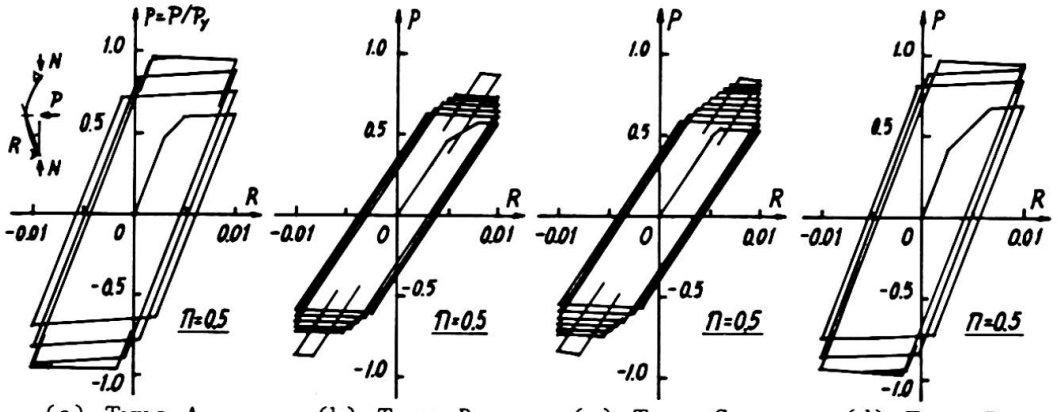


(a) Type A (b) Type B (c) Type C (d) Type D

Fig.6 Bending Moment-Curvature Relationship

2.3. Load-Deformation Relationship

Fig.7 shows the lateral load-deflection relationship of the column, the slendreness ratio of which is 25, under the constant axial load of $N=0.5N_y$. Alternately repeated horizontal force $P=pP_y$ is applied at the constant lateral



(a) Type A (b) Type B (c) Type C (d) Type D

Fig.7 Load-Deformation Relationship

sway rotation angle of R=±0.01. Figs.(a) and (b) correspond to the homogeneous member and Figs.(c) and (d) correspond to the hybrid one. In the case of the columns having a mild steel flange (type A and D), the load-deflection relationships converge to the steady state loop at the few cycles because of the small yield ratio (ζ=0.64) and of the large strain hardening coefficient (μ=1/110). The relationships of type B and C columns having high strength steel flange converge to the steady state loop at 17 and 13 cycles,respectively.

2.4. Variation of Maximum Strength and Accumulated Plastic Strain

Fig.8 shows the relationship between the maximum strength and the number of cycles. The abscissa W shows the number of half cycles. The maximum strength increases with the number of cycles. The strength of the hybrid member using a high strength steel as the web (type D) is the largest and using a mild steel as the web is the smallest. Convergence to the steady state loop is rapid in the case of the member having mild steel flange. Fig.9 shows the variation of the accumulated plastic strain of the web. The accumulated plastic strain induced is large in the case of a hybrid member using the mild steel web, and small in the case of the member using high strength steel web. In some cases, the plastic strain increases with the number of cycles even after the maximum strength converges to the steady state value.

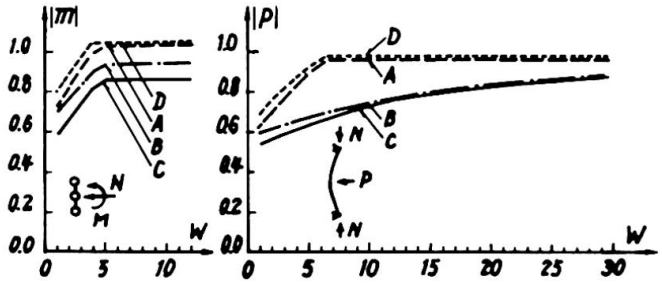


Fig.8 Variation of Maximum Strength

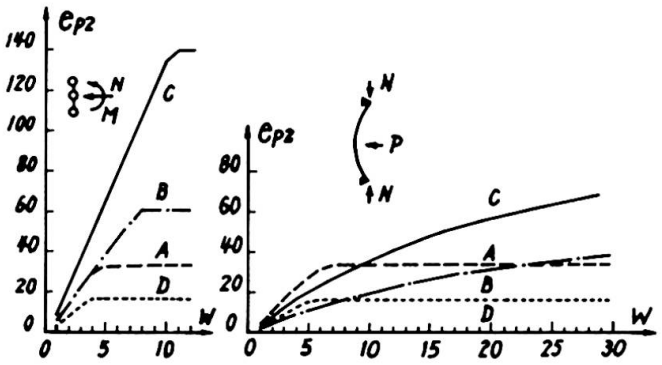


Fig.9 Accumulated Plastic Strain

3.CONCLUDING REMARKES

Elastic plastic behavior of the wide flange columns with homogeneous and hybrid sections was discussed. In the case of the members having high strength steel with large yield ratio and small strain hardening coefficient, the increase in the strength under the cyclic loading is relatively small. The hybrid member having a mild steel web shows a relatively small increment in the strength under the cyclic loading and induces the large amount of accumulated plastic strain in the web even under the small axial force range.

On the other hand, the hybrid member having a high strength steel web shows a relatively large increment in the strength and induces relatively small amount of plastic strain in the web.

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SUMMARY

Elasto-plastic cyclic bending deformation behaviour of the hybrid members is discussed assuming a wide flange section as a three points model and stress-strain relationship as tri-linear type. In the case of the hybrid member, the increase in the strength under the cyclic loading are larger and the amount of the accumulated plastic strains are smaller for the member with weaker flange than with weaker web. The limiting axial force level within which the bending resistance of the column asymptotes to that of pure bending is lower for the weaker web member.

RESUME

Les auteurs présentent le comportement élasto-plastique d'éléments hybrides soumis à des flexions cycliques; ils remplacent le profilé à larges ailes étudié par un modèle à trois surfaces et admettent une relation contraintes-allongements du type trilinéaire. Pour des éléments hybrides possédant un âme en acier à haute résistance, l'augmentation de résistance sous charges cycliques est plus élevée et la quantité de déformations plastiques accumulées, plus faible que pour une âme en acier doux. L'effort normal limite permettant asymptotiquement un moment de ruine égal à celui correspondant à la flexion pure est moins élevé lorsque l'âme est en acier doux.

ZUSAMMENFASSUNG

Das elasto-plastische Biegeverformungsverhalten einer zyklisch beanspruchten hybriden Stahlstütze ist unter der Annahme eines drei Punkt-Modelles des Breitflansc Querschnittes und einer tri-linearen Vereinfachung der σ-ε-Beziehung untersucht. Im Fall von hybriden Stahlstützen mit schwächerem Flansch ist der Zuwachs des Widerstandes grösser und die Verzerrungsakkumulation kleiner als für Querschnitte mit schwächerem Steg. Der Grenzwert des Normalkraftniveaus,für das der Biegewiderstand mit demjenigen unter reiner Biegung asymptotisch übereinstimmt, ist niedriger für einen Querschnitt mit schwächerem Steg.

On Structural Behaviour of Hybrid I-Beams

Sur le comportement à la ruine des poutres en I hybrides

Zum Tragverhalten hybrider I-Balken

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This contribution refers to Theme Va of the Introductory and Preliminary Reports. It represents certain proposal for the plastic analysis of hybrid I-beams assuming combined action of bending and shear.

Author's treatment of mentioned problem bases upon that suggested by Strel'bitskaya for homogeneous beams /1/. Simultaneously Schilling's optimum design criteria are taken into account /2/. Collapse loads determined are compared with those resulting from the ASCE-AASHO regulations /3/.

Plastic bending-shear interaction analysis of homogeneous I-beams bases generally on the yield condition

$$\sigma^2 + 3\tau^2 = \sigma_0^2 , \tag{1}$$

where σ , τ are the elastic bending and shear stresses, respectively, and σ_0 is the yield stress. According to /1/ the elastic stress patterns are as shown in Fig. 1; both of them are more (σ) or less (τ) inaccurate but fulfil the yield condition (1) at each point of the I-section and develop sufficiently exact (conservative) collapse load values. Similar analysis of hybrid I-beams can be based on stress patterns as given in Fig. 2.

With the specifications of Fig. 2 the fully plastic moment and shear, M_o and Q_o respectively, can be expressed as follows:

$$M_o = \sigma_o \frac{A_f h}{2} + \alpha \sigma_o \frac{th^2}{4} = \sigma_o \frac{th^2}{4}(1+\alpha), \tag{2}$$

$$Q_o = \alpha \tau_o th = \sigma_o \frac{th}{\sqrt{3}} \alpha , \tag{3}$$

whereby the following numerical values of α and σ_o are being considered:

$\alpha = 1.00$, for $\sigma_o = 36{,}000$ psi (homogeneous beam),
$\alpha = 0.72$, for $\sigma_o = 50{,}000$ psi
$\alpha = 0.36$, for $\sigma_o = 100{,}000$ psi (hybrid beams).

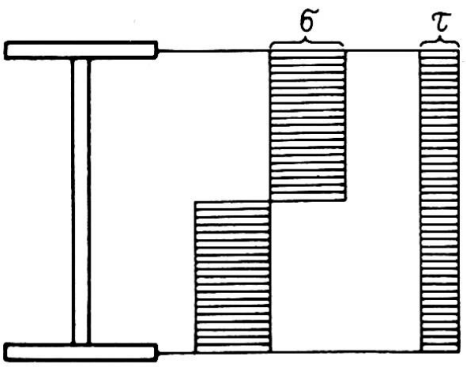


Fig. 1. Elastic stress patterns for homogeneous I-beam; assumed in [1]

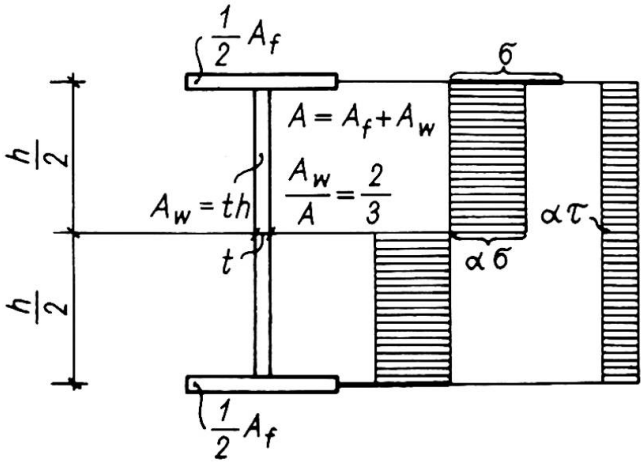


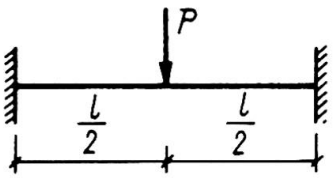
Fig. 2. Elastic stress patterns for hybrid I-beam; Author's assumption

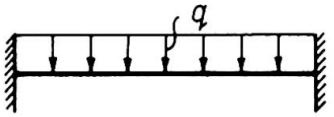
Investigations performed concern in particular the beam fixed at both ends carrying single load P at span-mid or uniform load q all over the span (Fig. 3).

According to Author

$$P = \frac{8\,\frac{M_o}{l}}{\sqrt{\frac{16\,M_o^2}{l^2\,Q_o^2}+1}} = \frac{8\,\frac{M_{ou}}{l}}{\sqrt{12\left(\frac{h}{l}\right)^2+\left(\frac{2\alpha}{1+\alpha}\right)^2}}$$

$$q = \frac{16\,\frac{M_o}{l^2}}{\frac{16\,M_o^2}{l^2\,Q_o^2}+1} = \frac{16\,\frac{M_{ou}}{l^2}}{6\left(\frac{h}{l}\right)^2\frac{1+\alpha}{\alpha}+\frac{2\alpha}{1+\alpha}}$$





According to ASCE-AASHO

$$P = \frac{16\,\frac{M_o}{l}}{\sqrt{64\,\frac{M_o^2}{l^2\,Q_o^2}\,\frac{\alpha}{1+\alpha}+1}+1} = \frac{8\,\frac{1+\alpha}{\alpha}\,\frac{M_{ou}}{l}}{\sqrt{12\left(\frac{h}{l}\right)^2\frac{1+\alpha}{\alpha}+1}+1}$$

$$q = \frac{32\,\frac{M_o}{l^2}}{\sqrt{128\,\frac{M_o^2}{l^2\,Q_o^2}\,\frac{\alpha}{1+\alpha}+1}+1} = \frac{16\,\frac{1+\alpha}{\alpha}\,\frac{M_{ou}}{l^2}}{\sqrt{24\left(\frac{h}{l}\right)^2\frac{1+\alpha}{\alpha}+1}+1}$$

Fig. 3. General expressions of collapse loads

Extending the interaction formula of /1/

$$\frac{M^2}{M_o^2} + \frac{Q^2}{Q_o^2} = 1 \tag{4}$$

over to hybrid I-beams and taking into account the corresponding equation

$$\frac{M}{M_o} + \frac{Q^2}{Q_o^2}\,\frac{\alpha}{1+\alpha} = 1$$

resulting from ASCE-AASHO regulations /3/ a set of general collapse load expressions can be obtained which is given in Fig. 3; therein

$$M_o = \frac{1+\alpha}{2\alpha}\,M_{ou} \tag{5}$$

where $M_{ou} \equiv M_o(\alpha=1)$ holds for homogeneous (uniform) beam.

On the basis of the expressions derived the corresponding interaction curves, in terms of Pl/M_{ou} and ql^2/M_{ou} respectively, as functions of the span-depth ratio l/h can be found; this is illustrated in Fig. 4 and Fig. 5.

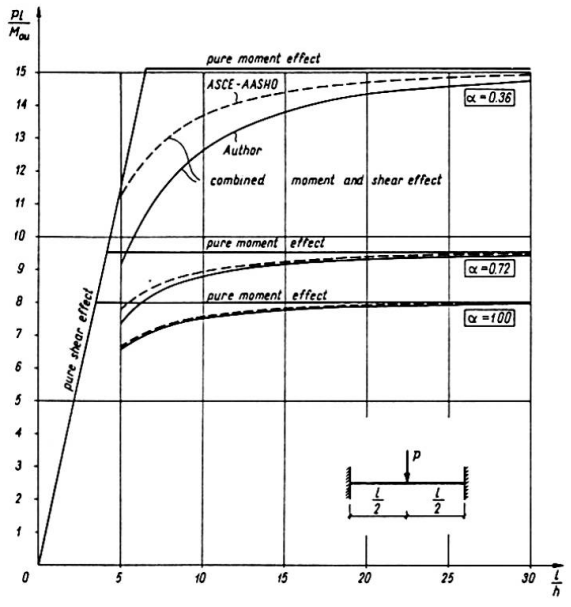


Fig 4 Collapse loads - concentrated

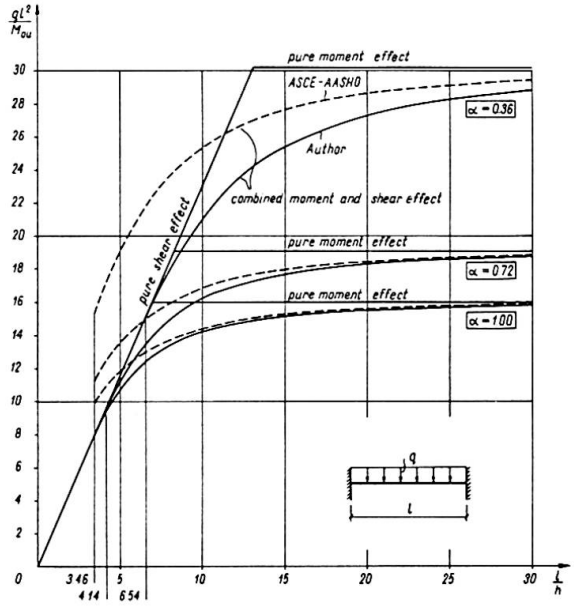


Fig 5 Collapse loads - uniform

In conclusion Author's proposal can be characterized as follows:

1. It yields collapse loads close (conservative) to those of the ASCE-AASHO regulations whereby the relative difference increases with a decrease of the l/h-ratio and the α-value.
2. It visualizes clearly the dominating effect of pure shear for small l/h-ratios.
3. It can be found handy analysing non-uniform plastic torsion problems (bimoment - flexural-torsional moment interaction, St.-Venant torsional moment effect) of hybrid sections, as shown for the homogeneous ones in /1/ and /4/.

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/4/ Strel'bitskaya, A.I.: Limit state of frames out of thin-walled members in bending with torsion (in Russian). Izdatel'stvo AN USSR, Kiev 1964.

SUMMARY

Simplified procedure is used to determine the ultimate load of hybrid I-beams in combined bending with shear. It has been found sufficiently exact and is recommended for analysis of similar problems within the theory of non-uniform torsion.

RESUME

Pour la détermination de la charge ultime de poutres hybrides en I soumises à la flexion et au cisaillement, on applique une méthode simplifiée. Celle-ci a donné des résultats suffisamment précis et est recommandée pour la résolution des problèmes similaires dans la théorie de torsion non uniforme.

ZUSAMMENFASSUNG

Ein vereinfachtes Verfahren wurde benutzt, um die Traglast hybrider I-Balken bei zusammengesetzter Biegung mit Schub zu bestimmen. Dieses wurde ausreichend genau gefunden und wird zur Behandlung benachbarter Probleme innerhalb der Theorie der Wölbkrafttorsion empfohlen.

Comportement à la fatigue des poutres hybrides raidies

Ermüdungsverhalten von ausgesteiften hybriden Trägern

Fatigue Behaviour of Stiffened Hybrid Beams

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Cette étude, financée par la CECA et le CRIF et réalisée sous la direction du Professeur R. BAUS, concerne des poutres hybrides dont les semelles sont en acier E 70 et dont l'âme et les raidisseurs sont en acier E 27 ; quelques poutres homogènes en acier E 70 ont également été testées. Les poutres comportent des raidisseurs soudés aux deux semelles, à l'exception d'une poutre hybride et d'une poutre homogène dont les raidisseurs ne sont soudés qu'à la semelle comprimée ; par ailleurs, une des poutres hybrides comporte une série de goussets soudés à la semelle tendue. Les dimensions générales des poutres et la mise en charge sont schématisées à la figure 1.

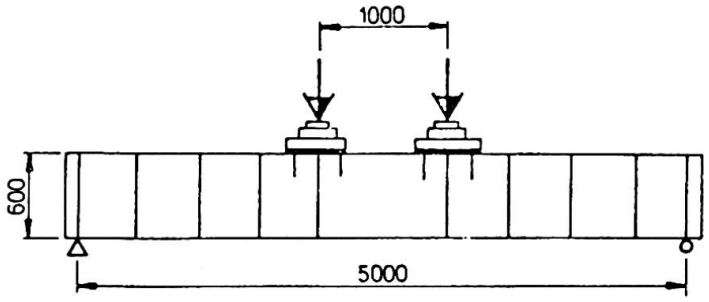
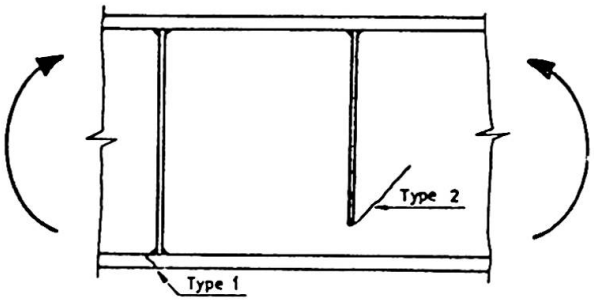


Schéma du montage d'essai

Figure 1

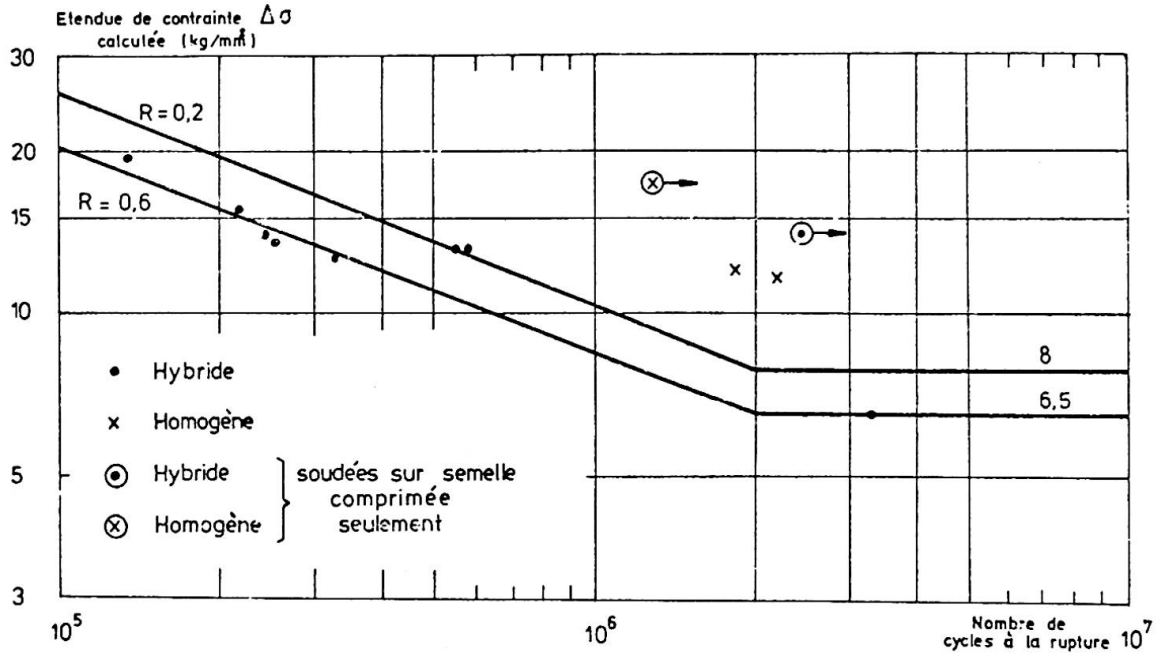
On diffuse les charges appliquées au moyen de plats de forte épaisseur, de plaques de néoprène et de goussets complémentaires. Les types de fissures observés sont présentés à la figure 2.



Types de fissure

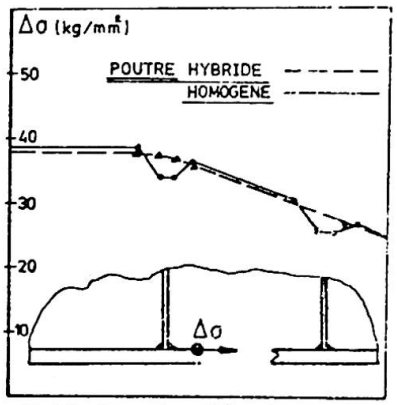
Figure 2

Les résultats des essais sont donnés à la figure 3, pour les fissures du type 1.



Courbes ΔO - N pour les fissures du type 1

Figure 3



Distribution des ΔO de service

Figure 4

On donne à la figure 4 la distribution des étendues de contraintes Δσ mesurées en service à la fibre intérieure de la semelle tendue entre les charges maximales et minimales. On constate une différence marquée entre la poutre hybride, dans laquelle la plastification initiale égalise les étendues de contrainte en service et la poutre homogène, dans laquelle le raidisseur reste un point dur engendrant une flexion locale.

Des essais effectués, on peut tirer les conclusions suivantes :

1° La résistance en fatigue correspondant à l'assemblage du raidisseur à la semelle est, dans le cas des poutres hybrides, voisine de celle des poutres homogènes classiques ;

2° Dans les poutres hybrides cette résistance ne paraît pas différente selon que le raidisseur est fixé aux deux semelles ou seulement à la semelle tendue, ou selon qu'il se trouve, ou non, sous la charge ; ceci peut être expliqué par l'égalisation des contraintes mentionnée plus haut ;

3° Dans les poutres hybrides, l'étendue des contraintes mesurées en service au niveau du cordon d'assemblage des raidisseurs est égale à la valeur calculée ; dans les poutres homogènes, elle est de 10 % inférieure à la valeur calculée. Ce fait et le meilleur profil des cordons réalisés sur les poutres homogènes expliquent leurs résultats légèrement supérieurs ;

4° De cette constatation, on peut conclure que les résultats des essais sur modèles de grande dimension doivent être étudiés avec circonspection, car les effets locaux du comportement structurel entraînent une différence entre les contraintes réelles et calculées au droit du cordon de soudure.

5° L'hypothèse selon laquelle, par suite de l'existence de contraintes résiduelles, il faut considérer que tout joint soudé d'une pièce soumise à fatigue travaille en traction entre unecontrainte maximale égale à la limite élastique de l'acier et une contrainte inférieure à celle-ci [1] constitue une simplification sécuritaire. Ni dans les essais effectués à Liège, ni en [2], il n'a été observé de fissuration au départ d'un joint de raidisseur à une semelle comprimée, même pour de grandes étendues de contraintes et en dépit de l'existence de contraintes résiduelles de traction particulièrement élevées (70 kg/mm2). On peut penser que la fissuration n'apparaît en zone comprimée qu'en présence d'actions locales de cisaillement ou de flexion [2], souvent ignorées lors du calcul des éléments.

6° Il convient d'éviter la soudure de raidisseurs sur la semelle tendue dans les zones de grands moments de flexion.

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RESUME

La résistance des poutres hybrides raidies aux sollicitations dynamiques est un peu inférieure à celle des poutres homogènes, en raison de certaines particularités du comportement structurel.

ZUSAMMENFASSUNG

Das Ermüdungsverhalten von ausgesteiften hybriden Trägern ist leicht weniger günstig als dasjenige von homogenen Trägern, wegen der besonderen örtlichen Spannungsverteilung in der Umgebung der Steifen.

SUMMARY

The resistance of stiffened hybrid beams to dynamic loads is a little less than that of homogenous beams, because of a particular stress distribution near the stiffener.

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Additional Tests for "Type 2 Crack" in Hybrid Girders

Essais complémentaires pour les fissures de "type 2" dans les poutres hybrides

Zusätzliche Versuche bezüglich der Risse von "Typ 2" in hybriden Trägern

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1. Introduction

At the Preliminary Report of the 10th Congress of IABSE[1)], we pointed out that the fatigue failure of thin-walled hybrid girders is generally due to "Type 2 Cracks" initiated at transverse stiffener-to-web fillet weldments. Further, a good correlation between the large-sized girder test results and the small-sized model test results using a steel plate with a transverse fillet welded attachment, was demonstrated. The girder tests, however, were not sufficient in numbers to warrant the fatigue strength, and the model tests did not exactly simulate the structural behavior of the girders because of the difference of strain states.

We are now presenting the results of additional fatigue tests, which have been carried out on axially loaded transverse non-load-carrying fillet welded specimens under strain control to examine the fatigue strength of "Type 2 Cracks" in hybrid girders.

2. Tests

Transverse non-load-carrying fillet welded specimens, as shown in Fig. 1, intended for simulating the boxing parts of the transverse stiffener-to-web fillet welds in hybrid girders, are axially loaded under strain control by fixing the out-put of strain gauges on the specimens to the maximum strain of 1700×10^{-6}, which corresponds to the design working strain of 80 kg/mm^2 high strength steel in a tension flange of girders.

The specimens consist of JIS-SS41 steel with a specified tensile strength of 41 kg/mm^2, and are divided into two series, namely, the ones of which at weld toe are as-welded and the others are lightly ground at the toe to examine the effect of finishing. Such a test simulates the structural behavior proper

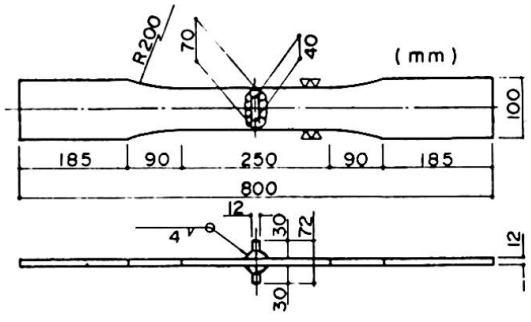

Fig.1 Details of As-Welded Specimen

to hybrid girders that a web reaches a kind of strain-controlled state after yielding due to the contribution of elastic frame action of flanges to the restraining of plastic flow of the web.

3. Results

The test results are graphically shown on the S-N diagram in Fig.2, in terms of equivalent stress range due to the strain range versus number of cycles to failure, neglecting the effect of stress ratios. The fatigue strength at 2×10^6 cycles of the as-welded specimens and of the lightly-ground specimens can be estimated at 9.5 kg/mm^2 and 10.5 kg/mm^2 as the mean value of stress ranges, respectively. The effect of finishing on the fatigue strength is not so remarkable at the present tests. This may be attributed to the fact that the grind operation was not fully performed and consequently some fine notches were not removed.

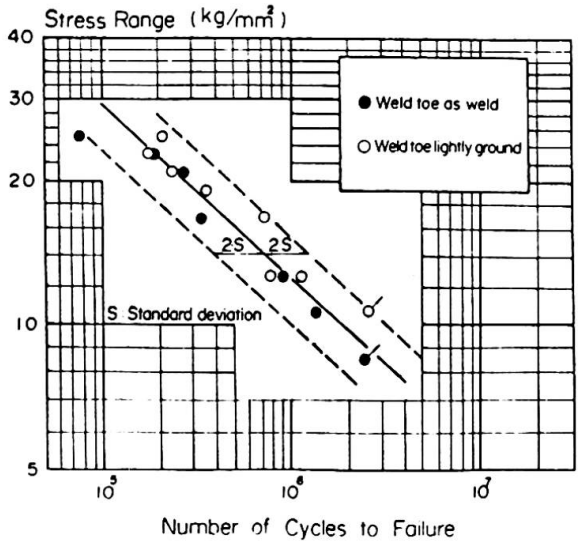

Fig.2 Fatigue Test Results

4. Discussions

(1) To study the influence of load controlling method on fatigue lives, comparison of the present test results with previous fatigue data on model specimens with the same configuration as the present ones, which were given by reanalysis of Gurney et al[2] about numerous tests, is shown in Fig.3.

It can be seen that the present test results, unexpectedly, agree well with the reanalyzed S-N curve by Gurney et al, which took into account the effect of residual stress on fatigue. It may be recognized that such a good agreement was obtained, because the correction of S-N curves was done on the assumption that residual stresses resulting from welding would generally be as high as the yield stress of the parent material as expected in a large as-welded structure, and such an assumption can be applied to the present tests,

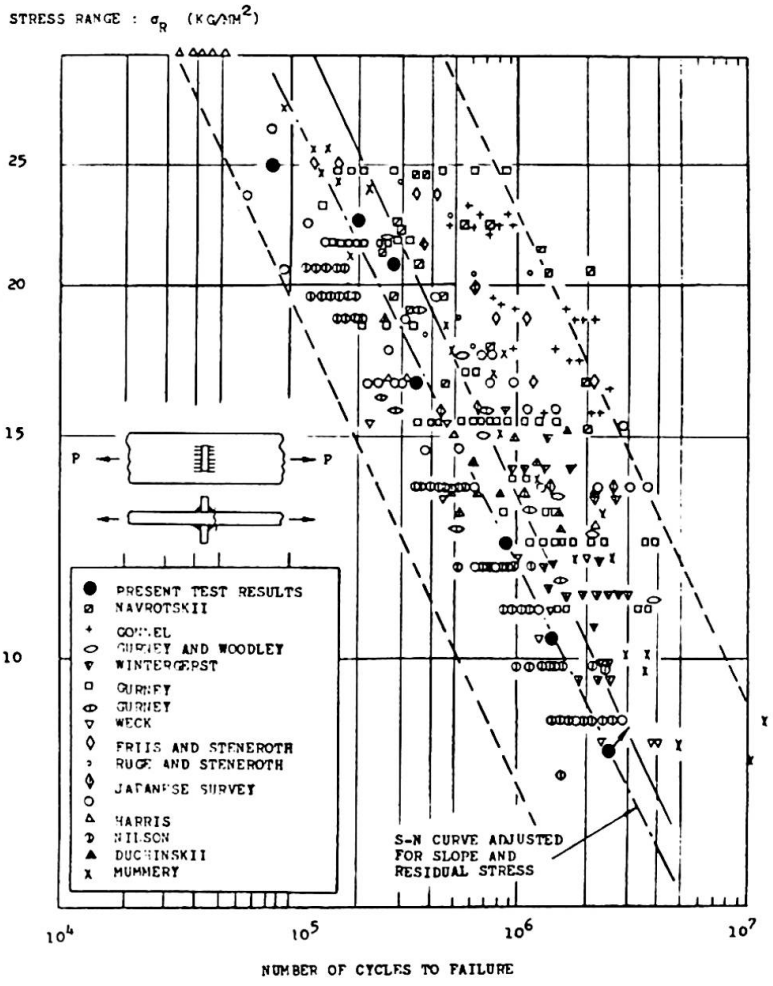

Fig.3 S-N Curves for Transverse Non-Load-Carrying Fillet Welded Joints

in which the specimens could yield at the maximum loading controlled by the strains.

(2) To study on the effect of steel grades on the fatigue strength, another comparison of the present test results for as-welded specimens with other steels[3], is made as shown in Fig. 4. Here, JIS-SM58 and WES-HT80 are high-strength steels with a specified tensile strength of 58 kg/mm^{2}and 80 kg/mm^2, respectively. The latter specimens are provided with the same configuration as the present test specimens, but their loading was controlled by loads from zero to tension.

Since any significant difference among those data cannot be observed, it will be concluded that, at such a welded joint with a large notch effect, the fatigue strength in terms of stress ranges are almost identical one another regardless of steel grades, stress ratios and load-controlling methods.

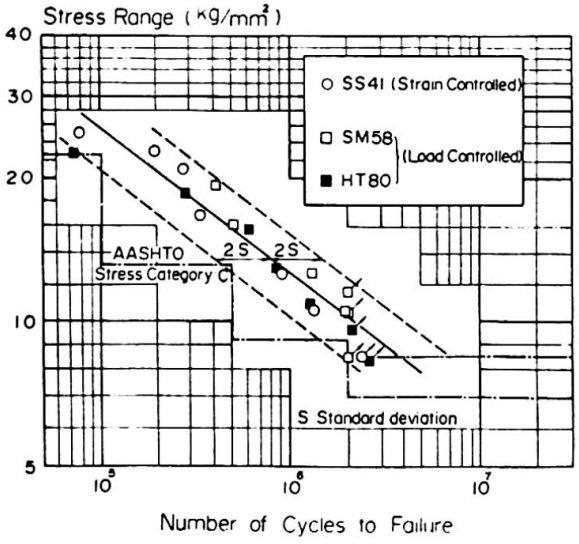

Fig.4 Fatigue Strength of Transverse Non-Load-Carrying Fillet Welded Joints for SS41, SM58 and HT80

(3) Finally, the relation between girder fatigue tests[1),4)] and some model fatigue tests is shown in Fig.5 . The model test results consist of the present test results and the previous ones in Japan [5),6)] on steel plates with transverse fillet welded attachments without boxing. The figure reveals that the present test results under relatively severe loading conditions, can be compared well with the lower 95 % confidence limit line of all the quoted test results except the present test results, and this implies that the present test results give conservative fatigue strengths due to "Type 2 Cracks" in hybrid girders.

It can be also noticed that the allowable fatigue stresses for "Stress Category C" specified at AASHTO Interim Specifications[7)], 1974, are reasonable for "Type 2 Cracks" in hybrid girders as seen in Fig. 4.

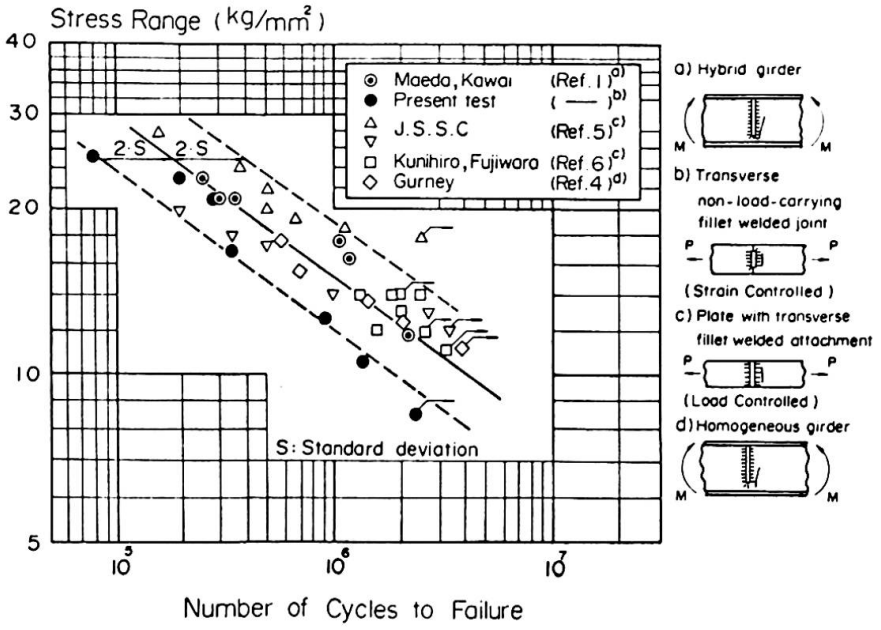

Fig.5 Comparison of Fatigue Strength for Type 2 Cracks in Girder Tests with Model Specimen Tests

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SUMMARY

To supplement the paper presented by the same authors at the Preliminary Report on "Structural Behaviour of Hybrid Girders in Bending and Application to Actual Bridges", additional tests on "Type 2 Cracks" are reported with the results and the discussions. It will be concluded that the fatigue strengths for Type 2 cracking are almost identical one another regardless of steel grades, stress ratios and load-control methods.

RESUME

Pour compléter la contribution présentée par les mêmes auteurs dans le Rapport Préliminaire, "Comportement à la flexion de poutres à âme pleine hybrides. Application aux ponts actuels", des essais complémentaires ont été réalisés pour étudier les fissures de"type 2". Résultats et discussions sont présentés. Il est possible de conclure que les résistances à la fatigue correspondant aux fissures de "type 2" sont pratiquement les mêmes, et indépendantes du type d'acier et de la contrainte moyenne.

ZUSAMMENFASSUNG

Um den von den gleichen Autoren im Vorbericht erschienenen Beitrag zu ergänzen ("Biegeverhalten von hybriden Vollwandträgern. Anwendung im Brückenbau"), wurden zusätzliche Versuche zur Untersuchung von "Typ 2" Rissen durchgeführt. Ergebnisse und Diskussion werden dargestellt. Die Schlussfolgerung lautet, dass die den Rissen "Typ 2" entsprechenden Ermüdungsfestigkeiten praktisch gleich sind, unabhängig von der Stahlfestigkeit und von der mittleren Spannung.

Fatigue Life Prediction of Hybrid Members

Prédiction de la résistance à la fatigue d'éléments hybrides

Vorhersage der Lebensdauer von auf Ermüdung beanspruchten hybriden Elementen

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1. INTRODUCTION

In the Preliminary Report for 10th Congress of the IABSE, 1976, fatigue crack growth rate observations were illustrated on the hybrid tee-shaped specimens to explore a possibility of the estimation of fatigue lives by using fracture mechanics(1).

Additional fatigue crack propagation test was conducted on a center-notched hybrid plate specimen to warrant the fatigue life prediction method for hybrid members.

2. FATIGUE CRACK PROPAGATION TEST

The configuration of the specimen is shown in Fig.1. One specimen consisting of JIS-SS41 (with specified tensile strength of 41 kg/mm^2) and WES-HW70 (with specified tensile strength of 80 kg/mm^2) was longitudinally butt welded along the specimen center line, and an artificial notch was introduced into its center to start a fatigue crack. The chemical analysis and mechanical properties of the materials are shown in Table 1.

The fatigue test was carried out in a 100-ton electro-hydraulic alternating testing machine under stress ranges of 15 kg/mm^2 and 10 kg/mm^2 and at a stress ratio of zero. Fatigue crack growth rate was observed on both the SS41 plate and the HW70 plate by using crack gauges and a 15 power magnifying glass with 1/20 mm micro scales.

The fatigue crack propagation results obtained, fatigue crack growth rate, da/dN, and stress intensity factor ranges, ΔK, were plotted in the conventional manner(2), that is, log-log plots of da/dN as a function of ΔK represented in the form of

$$\frac{da}{dN} = C\,(\Delta K)^m \qquad (1)$$

to evaluate the material constants, C and m, that characterize the fatigue crack propagation behavior of hybrid members.

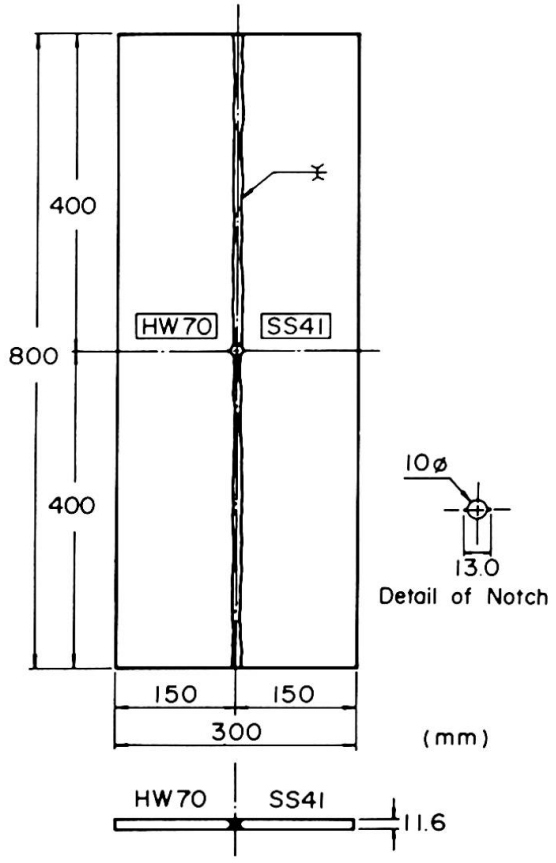
An effect of a slight difference of crack growth rate between SS41 and HW70 can be considered by using the Ishida's formula for an eccentrical crack in a finite plate (3) in calculating the value of K, based on the fact that the crack center shifted from the plate center as the crack grew. The eccentricity caused by the difference

of the crack growth rate, however, was small compared with the plate width, as was noted in previous report (1), such effect proved to be macroscopically negligible. The exposed fracture surface of the specimen is shown in Fig.2.

Table 1 Chemical Analysis and Mechanical Properties of Materials

Chemical Composition (Wt. %)											
	C	Si	Mn	P	S	Cu	Ni	Cr	Mo	V	B
WES-HW70	0.10	0.30	0.85	0.007	0.010	0.19	0.78	0.46	0.33	0.035	0.004
JIS-SS41	0.18	0.04	0.95	0.014	0.028	--	--	--	--	--	--

Mechanical Properties				
	Y.P.(kg/mm^2)	U.T.S.(kg/mm^2)	Elong.(%)	$_vE_{-5}$(kg·m)
WES-HW70	80	84	27	15.6
JIS-SS41	27	46	30	--



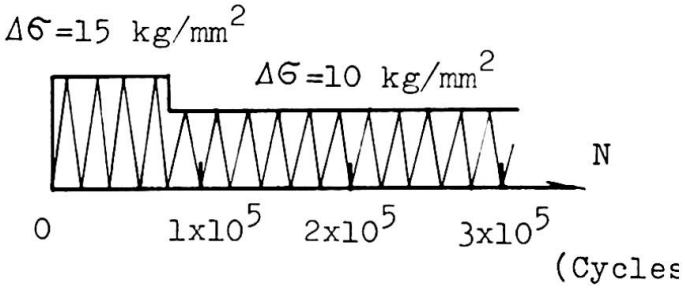
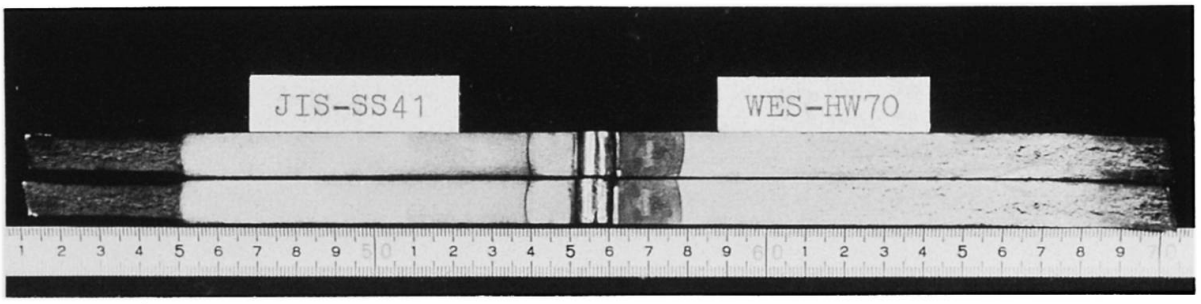


Fig.1 Configuration of Specimen and Stress Sequence

Fig.2 Exposed Fracture Surface



Fatigue crack growth rates, da/dN, in terms of Eq.(1) were shown in Fig.3, and the best fitted line by the method of least squares is also given in comparison with the derived crack-growth relationship given by Fisher et al.(4), a conservative upper bound for growth rates on ferrite-pearlite steels proposed by Barsom(5) and the authors' previous test results(1). The calculated material constants, C and m, were 1.2×10^{-11} and 3.6, respectively, namely the fatigue crack growth rate is numerically represented as

$$\frac{da}{dN} = 1.2\times10^{-11}\times(\Delta K)^{3.6} \qquad (2).$$

3. FATIGUE LIFE PREDICTION

To predict the fatigue lives on the previous hybrid tee-shaped specimens(1), these material constants and the value of K for a disk-like penny shaped crack in infinite body (6),

$$K = \frac{2}{\pi}\sigma\sqrt{\pi a} \quad ,$$

can be used, and the estimated S-N curves are calculated from the integrated version of Eq.(2).

The calculated S-N curves are shown in Fig.4 for various initial crack sizes, a_i, together with the test results on the hybrid tee-shaped specimens. The figure reveals that the predicted S-N curve taking the initial crack size, a_i, as 0.5 mm agrees well with the test results. This crack size of 0.5 mm is the same as the previously assumed one(1).

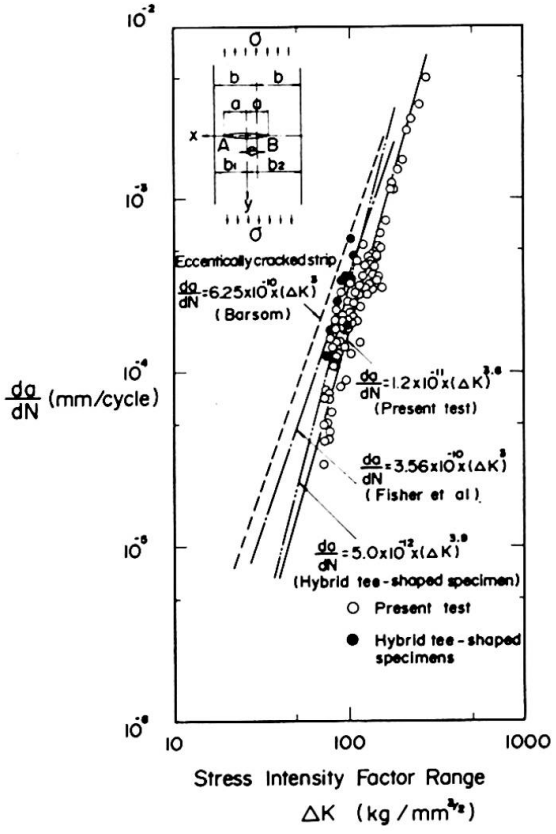


Fig.3 Summary of Fatigue Crack Growth Rate

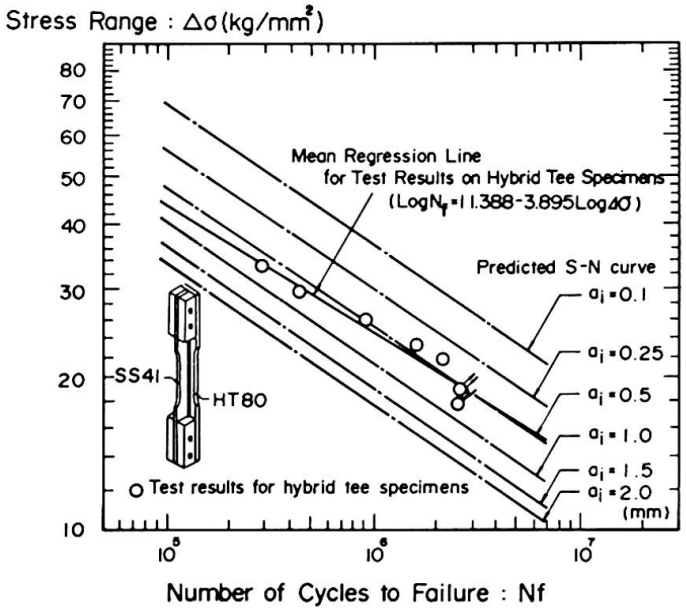


Fig.4 Comparison of Predicted S-N Curves and Previous Fatigue Test Results on Hybrid Tee-shaped Specimen(1)

4. CONCLUDING REMARK

The calculated S-N curve by using fracture mechanics, taking the initial crak size of 0.5 mm, successfully agreed well with the test results. Consequently, the fatigue life prediction method by fracture mechanics approach proved to be an effective tool for also hybrid members, if only a pertinent initial crack size can be evaluated.

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SUMMARY

A fatigue crack propagation test was conducted on a longitudinally butt welded 'WES-HW70' --'JIS-SS41' hybrid plate specimen to predict fatigue lives of hybrid members. The fatigue crack growth rates obtained were represented as a function of stress intensity factor ranges in the form of the so-called Paris' law. By using the material constants and fracture mechanics approach, the fatigue lives were successfully estimated on the hybrid tee-shaped specimens which were previously tested.

RESUME

La propagation d'une fissure due à la fatigue a été étudiée sur une plaque hybride en WES-HW70/JIS-SS41 soudée bout à bout, afin de prédire la résistance à la fatigue. Le développement de la fissure ainsi obtenue a été représenté en fonction des facteurs de concentration de contraintes, selon la loi dite de Paris. Au moyen des constantes des matériaux et de la mécanique de fissure, on a pu déterminer la résistance à la fatigue d'échantillons en forme de T qui avaient fait auparavant l'objet d'essais.

ZUSAMMENFASSUNG

Die Ausbreitung eines Ermüdungsrisses wurde an einer stumpf geschweissten hybriden Platte aus WES-HW70/JIS-SS41 untersucht, um daraus die Lebensdauer von hybriden Elementen vorhersagen zu können. Die gewonnene Rissausbreitungsgeschwindigkeit wurde in Funktion des Spannungskonzentrationfaktors nach dem sogenannten Paris'schen Gesetz dargestellt. Ausgehend von den Materialkonstanten lieferte die Bruchmechanik eine Vorhersage für die Lebensdauer von T-förmigen hybriden Proben, die vorher experimentell untersucht worden waren.

The Application of J_{IC} Fracture Criterion to the Fracture of Connections in Steel Structures

Application du critère de rupture J_{IC} au comportement à la rupture d'assemblages en construction métallique

Anwendung des Bruchkriterions J_{IC} auf das Bruchverhalten von Verbindungen im Stahlbau

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1. INTRODUCTION

The criterion based on critical stress intensity factor K_{IC} is applicable to brittle fracture in low stress level, and the design procedure has been satisfactorily established on this account. While, much of our concern is now paid to the fracture in high stress level such as the case of general yielding in the section of members. One of the examples of this kind is the transition fracture of heavy members of framework at room temperature reported by Prof. Ben Kato.[1)]

As one of the nonlinear fracture mechanics, the COD criterion is introduced in the introductory report, and strain around crack tip has been related to crack opening displacement by F.M.Burdekin.[2)] The COD theory, however, reveals to be indefinite in its physical concept when applied to wider yield portion. On the other hand, comprehensible is the physical concept of J_{IC} fracture criterion[3,4)] which is the extension of the concept of energy release rate G to nonlinear region, and because J_{IC} is comparatively insensitive to the scale effect, the specimens to obtain J_{IC} may be made smaller than those for K_{IC}.

In discussing the basic applicability of J_{IC} fracture criterion, J_{IC} value of SM50B(B class) and SM58QT(C class) are obtained from the 3-point bending test at room temperature.

Bending tests are carried out for the specimens of 4 types in various stress distributions,[5)] and the fracture configurations are discussed based on J_{IC} fracture criterion.

2. COMPARISON OF J_{IC} VALUES BETWEEN SM50B AND SM58QT

2.1 Specimens and Experimental Procedure

J_{IC} values are obtained from the 3-point bending test for which specimens are made from steel plates 25mm of SM50B and SM58QT. Chemical composition, mechanical properties and Charpy values of the materials are shown in Table-1~3 respectively.

The shape of the specimens, as shown in Fig.-1, is based on ASTM E399-72 and BS DD-19. All specimens are prepared with

fatigue crack at the tip of the crack. Three kinds of crack length, 23mm, 25mm and 27mm are adopted for the specimens of SM50B. The crack length for the specimen of SM58QT is either 23mm or 25mm.

The bending test is carried out at room temperature(+20°C) and -65°C. Since it is difficult to determine the displacement δ_c experimentally to initiate crack propagation at room temperature, the crack propagation is checked by cutting the specimens in the cource of experiment after J. A. Begley and J. D. Landes.[6]

The value of J integral is obtained approximately from the following equation.

$$J = \frac{2}{Bb}\int_0^{\delta_i} Pd\delta \qquad (1)$$

where, B: Width of the specimen
b: Difference of crack length a from height W
P: Load
δ: Displacement at the loading point

2.2 Test results

Relationships between J value and increment of crack length Δa are shown in Fig.-2 which are lead by P~δ diagrams in the bending test. The results from the experiments on various crack lengths present reasonable J_{IC} values. It is recognized, therefore, that the method proposed by Begley and Landes for determining J_{IC} value should be available.

Fig.-3 shows a comparison of J~Δa relationships between SM50B and SM58QT obtained from the bending test.

In the study by J. R. Rice, et al.[7], J value is calculated for simplified P~δ relationships composed of elastic part and perfectly plastic part beyond general yielding. In this paper, P~δ relations are computed up to the general yield point by applying finite element method. After reaching the general yield point, P~δ relation is assumed as perfectly plastic. Fig.-4 shows the element division of the specimen. Calculated values of J integral are compared with the experimental results in Fig.-5.

It is confirmed that the above method for estimation of J value by calculation should be available.

It is observed in Table-4 that J_{IC} value of SM58QT is considerably smaller than that of SM50B at room temperature. Seeing that J_{IC} value of SM58QT at -65°C is larger than that of SM50B, the difference of J_{IC} values at room temperature should be adequately noted.

3. DISCUSSION CONCERNING THE FRACTURE OF TEE-JOINTS

It is emphasized in the introductory report that the welding condition free of micro cracks and steels possessing enough notch toughness should be substantial to prevent unstable brittle fracture. Further investigation should be focussed on the relations of notch toughness of material and fracture under complex stresses such as connections in steel structure.

Bending tests by using the specimens of 4 types are carried out in various stress states and conditions of test temperature. As shown in Fig.-6, the thickness of the specimens is 25mm and roller span is 280mm.

The specimen of J-type has the ratio of thickness to width 25:35 and this type is a model to expect plane stress. The ratio

Table-1 Chemical Composition (%)

Grade of Steel	C	Si	Mn	P	S	Cu	Cr	Nb	V
SM50B PL25mm	0.16	0.39	1.46	0.014	0.025	0.01	—	—	—
SM50B PL65mm	0.16	0.46	1.45	0.008	0.007	0.31	0.03	0.036	—
SM58QT PL25mm	0.12	0.27	1.11	0.012	0.007	—	0.16	—	0.03

Table-2 Mechanical Properties

Grade of Steel	Y.P.	T.S.	EL.	φ
SM50B PL25mm	41	60		73
SM50B PL65mm	39	56	38	77
SM58QT PL25mm	55	71	30	71

Y.P.: Yield Point (kgf/mm²)
T.S.: Tensile Strength (kg/mm²)
EL.: Elongation (%)
φ: Reduction of Area (%)

Table-3 Charpy Values

Grade of Steel		$_vE_0$	$_vT_{rE}$	$_vT_{rS}$
SM50B PL25mm	Surface	15.1	-6	-7
	(1/2)t	12.5	-12	-10
SM50B PL65mm	Surface	30.0	-67	-65
	(1/2)t	30.0	-52	-54
SM58QT PL25mm		24.0 *		

$_vE_0$: Absorbed energy at 0°C, but * is at -5°C (kgf.m)
$_vE_{rE}$: Energy transition tempetrature (°C)
$_vE_{rS}$: Surface transition temperature (°C)

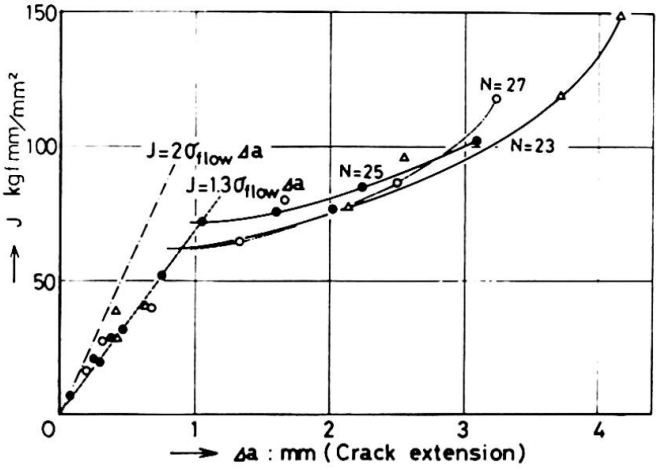

Fig.-2 Relations between J and Δa (SM50B)

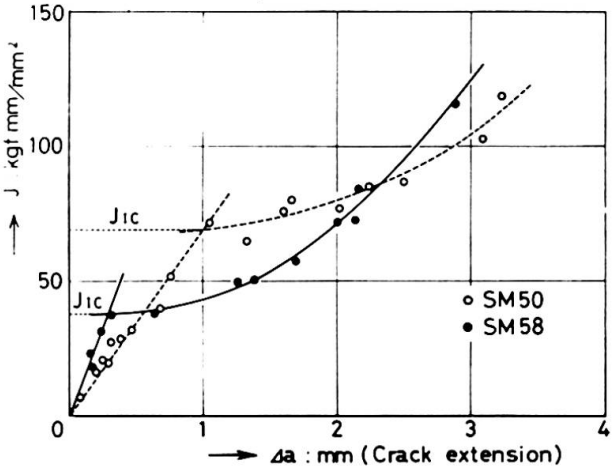

Fig.-3 A comparison of J~Δa relations between SM50B and SM58QT

Table-4 J_{IC} (kgf.mm/mm²) values

Grade of Steel	Test Temperature 15°C	-65°C
SM50B pl25	63	12.8
SM58QT PL25	43	15.1

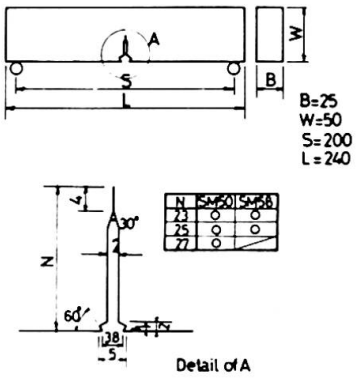

Fig.-1 Specimens for the 3-point bending test

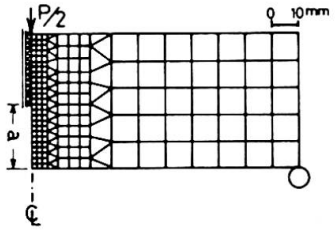

Fig.-4 Element division

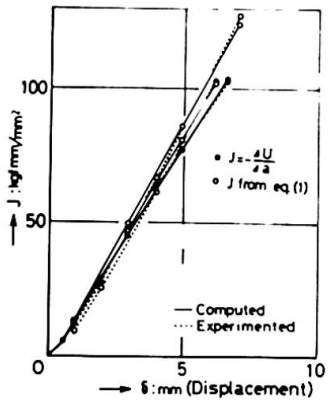

Fig.-5 Relations between J and δ

of M-type is 25:125 to expect plane strain state. B_T-type is made of steel plate 25mm with a Tee-joint where is welded. Tee flange of this type is the same dimension as M-type. The specimen of A_T-type is shaped from steel plate 65mm in order to eliminate the influence of welding. The dimension of A_T-type is same as that of B_T-type except fillet portion. The fillets of A_T and B_T-type are shown in detail in Fig.-7, respectively. The radius r at the toe of fillet in A_T-type specimen is determined to be 2mm in order to approach the same condition as the welded part in B_T-type. B_T specimen is built up by manual welding and stress relief annealing is not treated after welding. A_T and B_T-type are both pull-bend specimens.

The specimens of J-type, M-type and B_T-type are made from the steel plate which is dealt with at the preceding section. The properties of SM50B plate 65mm for A_T-type are also shown in Table-1~3. J_{IC} values for these mother metals and welded portion of the joint are summarized in Table-5.

Fig.-8 shows the relationships between the maximum bending angle θ_{max}(degree) and the test temperature T°C obtained from these bending tests.[4] It is observed in this figure that the more complex the stress state is, the smaller θ_{max} is. Especially, the specimen of B_T-type with welded joint might be the most fragile in these 4 types.

J values of the specimens are estimated by the method described at previous section. Element divisions are shown in Fig.-9.[6] J values are obtained by computing two P~δ relationships of the cases a/W=0 and a/W=2/25. Fig.-10 shows the computed results of P~δ relationships.

J_C value for each specimen which is J value at fracture of the specimen is determined by θ_{max} and P~δ curves. The ratio of J_C to J_{IC} is shown in Table-5.

This result indicates that the more complex the stress distribution of the specimen is, the smaller J_C/J_{IC} is. This ratio comes to almost unit in the case of B_T.

4. Conclusion

J_{IC} values of SM50B and SM58QT both of which are obtained from the 3-point bending test at -65°C are remarkably equal, nevertheless J_{IC} value of SM58QT is approximately 2/3 of that of SM50B at room temperature.

Experimental results from bending tests on various stress states of 4 types specimens indicate that the more complex the stress distribution is, the more fragile the joint of steel structure is. And this phenomenon can be explained by J_{IC} fracture criterion.

Finally, it should be pointed out that the independence of path in J integral is valid only under the deformation theory, and it is not directly applicable to the frames in which stress redistribution may take place significantly. J integral in this paper is introduced beyond limitation in this sence. It is necessary, therefore, to obtain more basic experimental data in order to apply J_{IC} fracture criterion in practice.

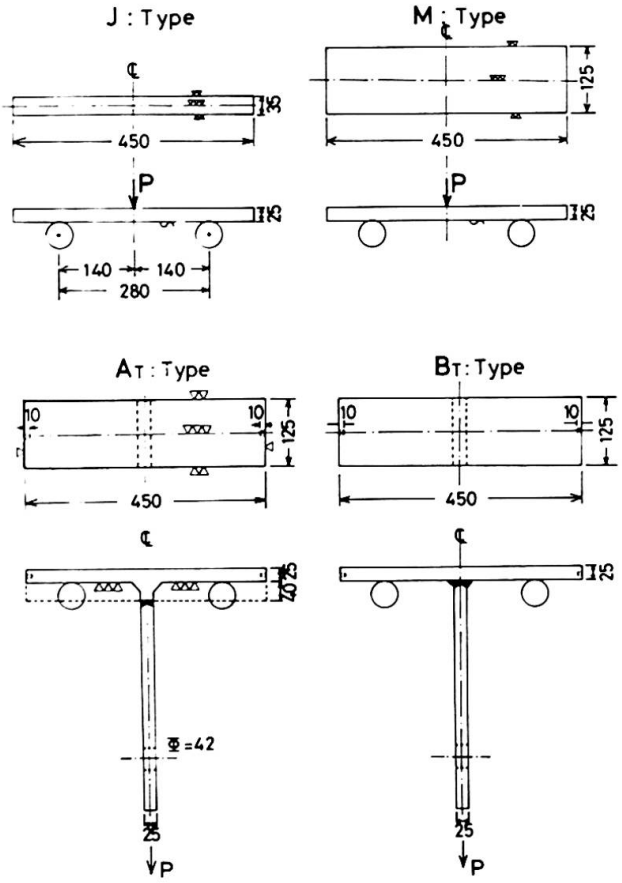

Fig.-6 Specimens of 4-types for bending tests

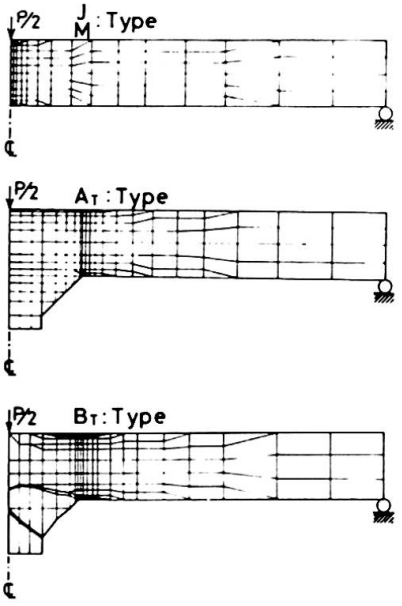

Fig.-9 Element division for 4-type specimens

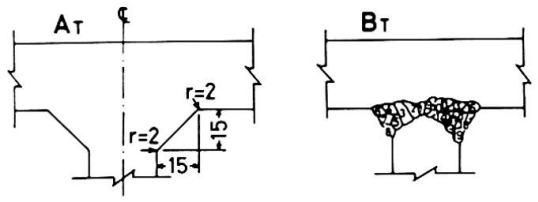

Fig.-7 Details of the fillets

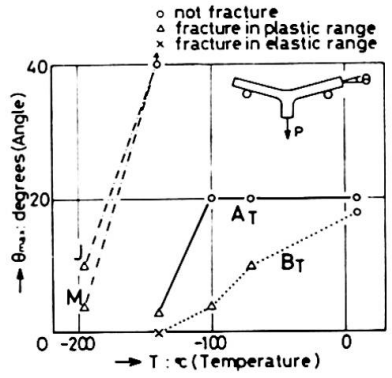

Fig.-8 Test results

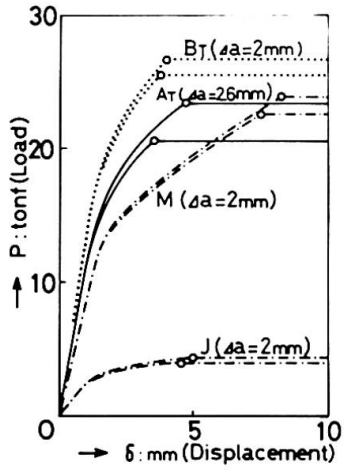

Fig.-10 Computed results

Table-5 J_C Value and the Ratio to J_{IC}

	J-type	M-type	A_T-type	B_T-type
$J_C(\frac{kgf.mm}{mm^2})$	35.46	8.42	143.11	30.38
$J_{IC}(\frac{kgf.mm}{mm^2})$	0.49	0.49	76.24	26.80
J_C/J_{IC}	72.4	17.2	1.88	1.13

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SUMMARY

J_{IC} values are compared between SM50B and SM58QT. Both are remarkably equal at -65° C, nevertheless J_{IC} value of SM58QT is approximately 2/3 of that of SM50B at room temperature. Bending tests on various stress states of 4-types specimens are carried out, and the experimental results indicate that the more complex the stress distribution is, the more fragile the joint of steel structure is. This phenomenon can be explained by J_{IC} fracture criterion.

RESUME

Les valeurs de J_{IC} sont comparées pour les aciers SM50B et SM58QT. Elles sont remarquablement identiques à - 65° C, bien que,à la température ambiante, la valeur J_{IC} du SM58QT ne soit que les 2/3 de celle du SM50B. Des essais de flexion sous différents états de contraintes ont été effectués sur des éprouvettes de 4 types. Les résultats expérimentaux montrent que, plus la distribution de contraintes est complexe, plus les assemblages sont fragiles. Ce phénomène peut être expliqué à l'aide du critère de rupture J_{IC}.

ZUSAMMENFASSUNG

Die J_{IC} Werte werden für die Stähle SM50B und SM58QT verglichen. Beide sind praktisch gleich bei -65° C, obwohl bei Raumtemperatur der J_{IC} Wert des SM58QT nur ungefähr 2/3 des Wertes des SM50B beträgt.Biegeversuche unter verschiedenen Spannungszuständen wurden an Proben mit 4 Ausbildungsformen durchgeführt; die experimentellen Ergebnisse haben gezeigt, dass die Gefahr eines Sprödbruches bei Stahlbauverbindungen mit der Komplexität der Spannungsverteilung zunimmt. Dieses Verhalten wird mit dem Bruchkriterium J_{IC} erklärt.

Further Studies on Composite Beams with Formed Steel Deck

Etudes complémentaires sur des poutres mixtes à platelage métallique

Weitere Untersuchungen über Verbundträger mit Stahlblechdecken

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After publication of our paper in the Preliminary Report, Tenth Congress IABSE (1), additional tests were undertaken on composite beams with 3 in. (76 mm) formed steel deck with a w/h ratio equal or greater than 2.0 In addition, it was desirable to provide further experimental data on shear connectors with an increased embedment length above the rib. In the study reported in Ref. 1, most stud shear connectors were embedded 1½ in. (38 mm) above the rib (H - h). In the additional beam tests, all connectors were embedded 2 in. (50.8 mm) above the rib in order to optimize the shear connector resistance. The rib geometry provided w/h ratios of 2 and 2.42.

As a result of the additional beam tests, the shear connector resistance was re-evaluated and a slight modification suggested by Lim (2) was introduced into Eq. 2 of the preliminary report. It was found that replacing the coefficient 0.6 with $0.85/\sqrt{N}$ provided a better fit to the test data with single connectors in a rib, as well as providing for more than two connectors in a rib. This results in the expression:

$$Q_{rib} = \frac{0.85}{\sqrt{N}} \cdot \frac{H - h}{h} \cdot \frac{w}{h} \cdot Q_{sol} \leq Q_{sol} \qquad (1)$$

where N = number of shear connectors placed in a rib. The parameters w, h and H were defined previously (w = average rib width, h = height of rib, H = height of stud shear connector) and Q_{sol} is the connector strength in a solid slab.

Figure 1 shows the 56 beam tests results summarized in Fig. 4 of Ref. 1 as well as 19 additional beam tests including one test with three connectors per rib. The plot shows that Eq. 1 provides a good estimate of flexural capacity for 1, 2 or 3 connectors in a rib. Pushoff tests summarized in Ref. 3 indicate the same type of change in connector capacity with varying numbers of shear connectors in a rib. Equation 1 provides the same connector capacity as Eq. 2 in the preliminary report when two connectors are placed in a rib. For a single connector a higher capacity results which approaches the solid slab value for longer embeddment lengths.

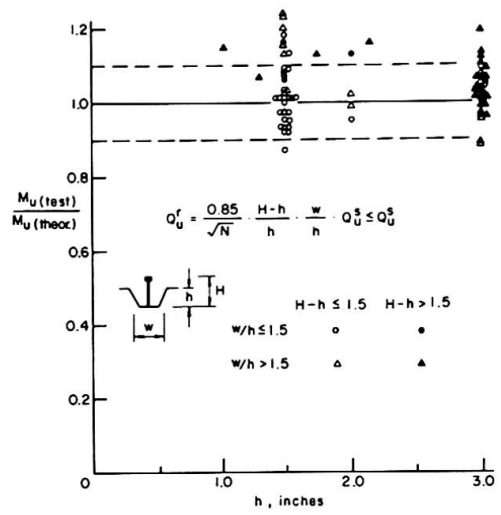

Fig. 1

In the Preliminary Report, Eq. 4 was incorrectly shown. It was intended to be expressed as:

$$I_{eff} = I_s + \sqrt{\frac{V'h}{Vh}} (I_{tr} - I_s) \quad (2)$$

The shear connection ratio was missing its exponent in the Preliminary Report.

All additional tests verified the applicability of Eqs. 1 and 2 to composite beams with formed steel deck, including deck with larger w/h ratios.

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SUMMARY

Additional tests on composite beams with formed steel deck verified that the w/h ratio could be extended beyond 2. A modification to the expression developed for shear connection capacity was found to provide for differing numbers of shear connectors installed in a rib.

RESUME

Des essais complémentaires sur des poutres mixtes avec platelage métallique ont montré que le rapport w/h (largeur de la nervure/hauteur de la nervure) peut être étendu au-delà de 2. Une modification de l'expression développée pour la résistance des connecteurs a permis de tenir compte du nombre de connecteurs par nervure.

ZUSAMMENFASSUNG

Zusätzliche Versuche an Verbundträgern mit Stahlblechdecken zeigten, dass das w/h Verhältnis (Rippenbreite/Rippenhöhe) über 2 hinaus vergrössert werden darf. Eine Erweiterung des Ausdruckes für die Dübeltragfähigkeit, die die Anzahl der Dübel pro Rippe berücksichtigt, ist gefunden worden.

The Ultimate Load of Monosymmetric Sections due to Lateral Torsional Buckling

Calcul de la charge ultime de sections monosymétriques en tenant compte du déversement

Traglasten von einfach-symmetrischen Profilen unter Berücksichtigung des Kippens

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1. INTRODUCTION

Structural members loaded by transverse loads and moments acting in the plane of greatest stiffness may also twist and deform laterally as shown in fig.1. The most general case includes initial imperfections of a bar or eccentricities of the loads and inelastic material behaviour, fig.2.

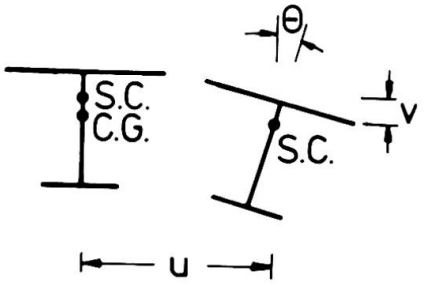

fig.1

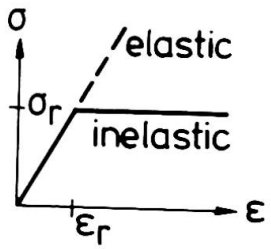

fig.2

Solutions for this problem of lateral torsional buckling are well known for the elastic behaviour, including monosymmetric sections. For inelastic material behaviour solutions are known only recently ([1] - [6]), most of them for double symmetrical sections. For monosymmetric sections only single results are known ([2], [4]).

2. THEORETICAL SOLUTION

The used theoretical solution is an approach for the inelastic material behaviour. It takes into consideration the elastic-plastic stress-strain relationship (fig.2), the spread of the plastic zones in the longitudinal direction of the beam-column, residual stresses and the alteration of the position of the shear centre due to plastification.

At first there is investigated a solution using the theory of elasticity. If the greatest stress in the most unfavourable fibre of the section exceeds the yield stress, this must be taken into account by iteration. Thus the spread of plastification in the section, the alteration of deformations and twist are considered. The ultimate strength is reached when equilibrium between external and internal forces is no longer possible.

This solution is described properly in literature ([2], [3], [4], [7]).

	St 37 σ_y=240 N/mm²	StE 70 σ_y=700 N/mm²
780·8 — 500·20 / 400·15 [mm]	1769	5159
780·8 — 400·20 / 400·20 [mm]	1828	5331

Table 1: Full plastic moments [kNm]

3. RESULTS

For getting numerical results, there are investigated a monosymmetric and a double symmetrical section with equal area and height. The loading is assumed as concentrated load, because the most unfavourable case of constant end moments are seldom given in practical members. The level of application of the load is the upper flange. The end conditions correspond to simple support in the lateral plane. This means that lateral deflection and twist are prevented but no resistence is provided against either lateral bending or warping.

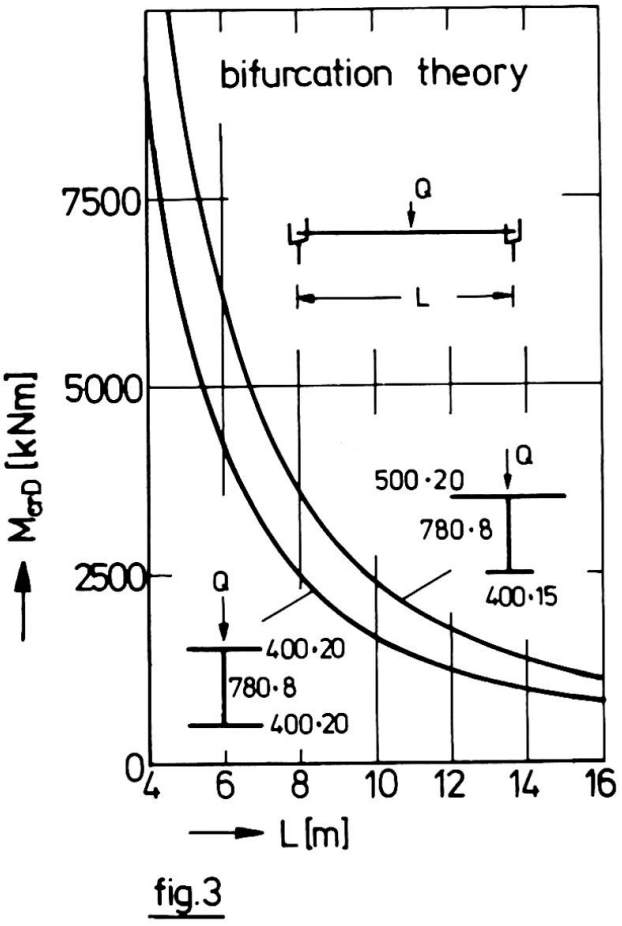

fig.3

The greatest capacity of the section is given by the full plastic moment, [8]. The results are given in table 1. It is seen, that the symmetric section gives the greater maximum capacity, but the difference does not exceed 3%.

The monosymmetric section is the better one, if the bifurcation theory is used (fig.3). The reason is, that the compression flange, which may buckle, get a greater area, the center of gravity and the shear center have a smaller distance to the compression flange and therefore it is clear, that the critical load increases. The differences amount to 45% related to the double symmetrical section.

The ultimate load carrying capacity is seen in fig.4. In addition mild steel St 37 and high strength steel StE 70 are compared. They are drawn in a nondimensional way. The parameters are M/M_{Pl} and $\bar{\lambda}$. M is the given moment of the beam, taking into account the load enhancement factor. M_{Pl} means the full plastic moment (table 1). The value of $\bar{\lambda}$ depends on the yield stress and the critical moment due to lateral torsional buckling. For each length of the beam $\bar{\lambda}$ can be calculated using table 1 and fig.3.

The value of the residual stresses was chosen to a maximum compression value of $\sigma_{rc} \cong 120$ N/mm².

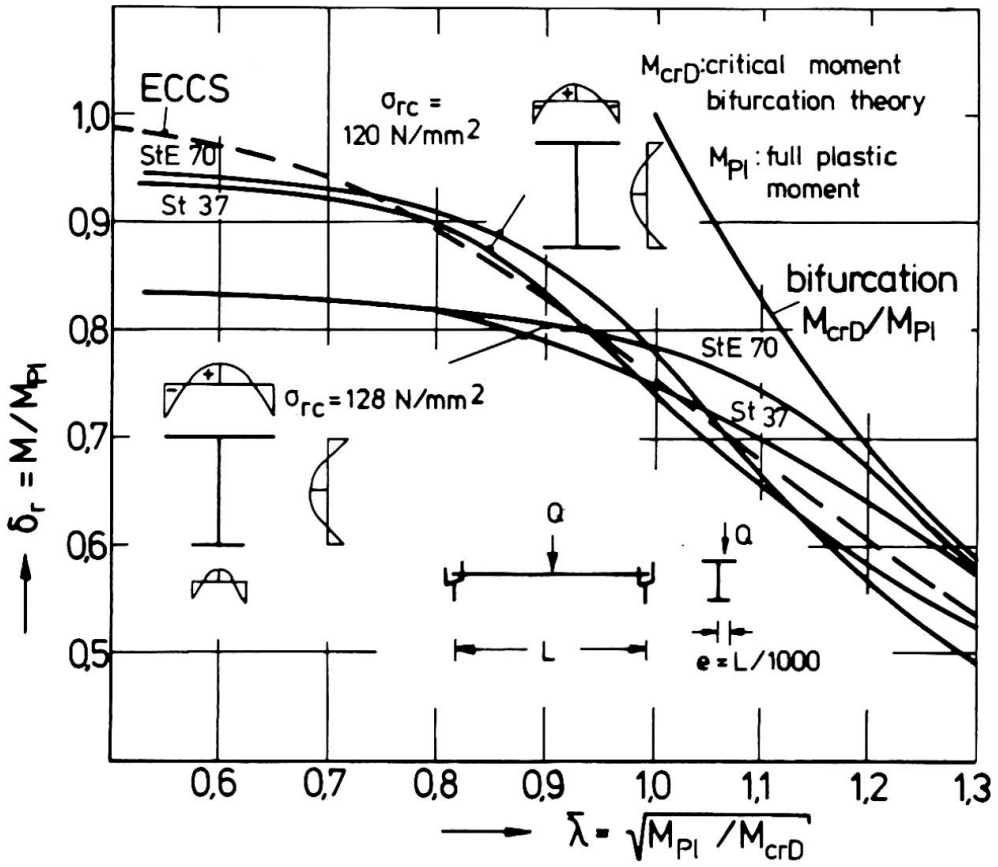

fig.4: Utimate load carrying capacity

That means for steel St 37 50% of the nominal yield stress. Even for the high strength steel StE 70 the same value for the residual stresses was chosen.

Some interesting results are seen in fig.4. It is seen, that for small values $\bar{\lambda} \lesssim 1{,}0$, that means small values L of the length, the ultimate load of the monosymmetric section lies significantly below that of the double symmetrical section. In the region of $\bar{\lambda} \gtrsim 1{,}0$ it is contrary, but the differences are a great deal smaller. The differences between St 37 and StE 70 are small, they exceed not more than 8%.

The curves for the double symmetrical section give a satisfying agreement to the new ECCS-curve [9], which should be valid only for double symmetrical sections. For the monosymmetric section the ECCS-curve may be used for $\bar{\lambda} > 1{,}0$.

In addition a comparison is made to the elastic theory of second order. This theory is usually used if there is no possibility to calculate the real ultimate strength. Using this theory the assumption is made that the load carrying capacity is reached when at the most unfavourable point of the beam the yield stress σ_y is reached.

In the present case there are 3 types of normal stresses taken into account. These stresses are originated from moments about the strong axis, moments about the weak axis and warping moments.

The used calculation method is the energy method. The computer program was kindly given by D.BAMM, it is similar to these discribed in [3], [4].

The results are given in fig.5. It is seen that the shape of the curves is similar using the ultimate strength method or the elastic second order theory.

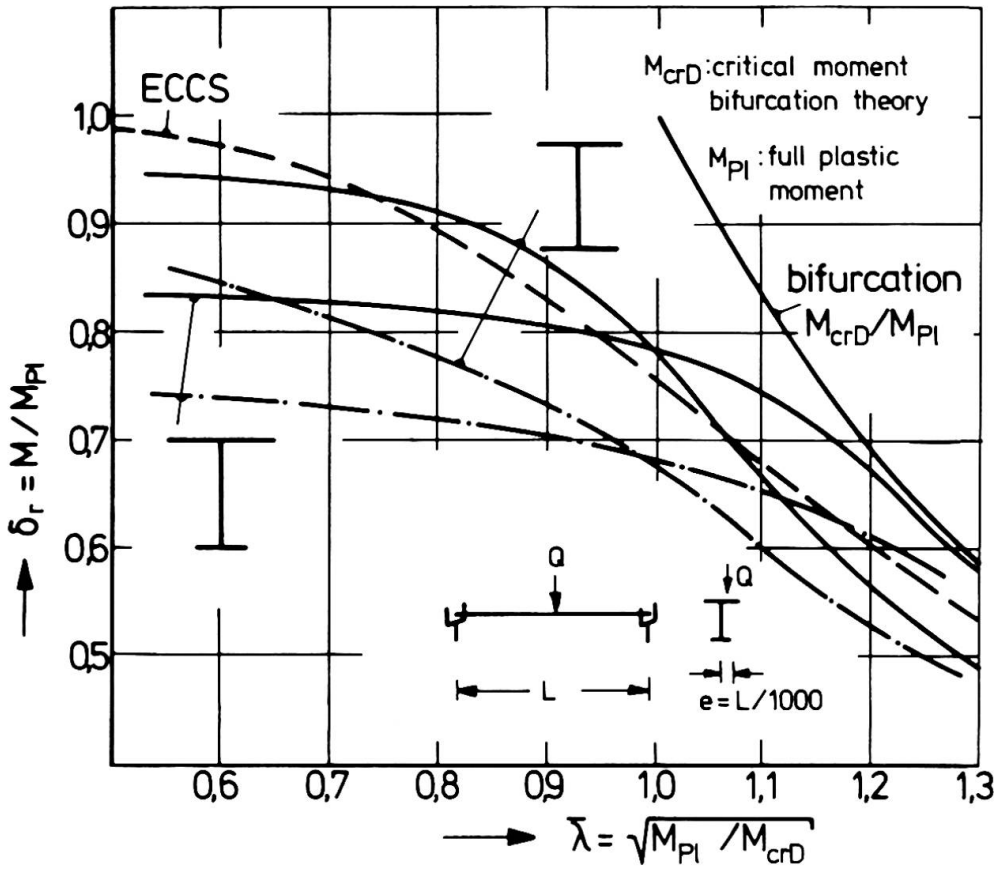

fig.5: load carrying capacity StE 70
—— ultimate strength
—·— second order theory, $\sigma_{cr} = \sigma_y$

But in the whole region of $\bar{\lambda}$ is a great reserve from reaching the yield stress at the most unfavourable point to the load carrying capacity, where the cross section is partly yielded.

Therefore from the economical point of view it is usefull to use the ultimate strength method or as an approximation using the method given in [6].

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SUMMARY

The paper deals with the load carrying capacity of monosymmetric sections taking into account lateral torsional buckling. Numerical results are calculated for a double symmetrical and a mono symmetrical cross section with equal height and area. From the results is seen, that the ultimate load of the monosymmetric cross section is smaller than that of the doublesymmetric cross section. The ECCS recommendation curve should only be used for monosymmetric cross-sections, if the length of the bar is great.

RESUME

L'article traite de la résistance ultime de sections monosymétriques en tenant compte du déversement. Des résultats numériques sont calculés pour des profils à symétrie double ou simple ayant des hauteurs et des surfaces égales. Les résultats montrent que la résistance ultime du profil monosymétrique dans le domaine plastique est inférieure à celle du profil à symétrie double. Pour des sections monosymétriques, les courbes des recommendations CECM ne doivent être appliquées que lorsque l'élancement est grand.

ZUSAMMENFASSUNG

Der Beitrag beschäftigt sich mit der Traglast von einfach-symmetrischen Profilen im Hinblick auf das Stabilitätsproblem des Kippens. Zahlenmässige Ergebnisse werden für ein doppelt-symmetrisches Profil und ein einfach-symmetrisches Profil mit gleicher Höhe und Fläche angegeben. Aus den Ergebnissen ist zu ersehen, dass im plastischen Bereich die Traglast des einfach-symmetrischen Profils unter der des doppelt-symmetrischen liegt. Die Bemessungskurve der EKS-Empfehlungen darf für einfach-symmetrische Profile nur benutzt werden, wenn die Stablänge gross ist.

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Utilisation d'aciers à haute résistance dans quelques ponts mixtes suisses

Verwendung hochfester Stähle im schweizerischen Verbundbrückenbau

Application of High-Strength Steels in some Swiss Composite Bridges

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Zurich, Suisse

1. Introduction

La réalisation du réseau des routes nationales suisses a entraîné, ces dernières années, la construction de nombreux ponts mixtes acier-béton. Une vingtaine d'entre eux comportent des tronçons en acier à haute résistance, la plus grande partie de l'ouvrage étant cependant exécutée en acier E 36[1)], c'est-à-dire dans la nuance la plus répandue pour les ponts-routes européens. Dans le cadre du septième congrès de l'AIPC [1] nous avions eu l'occasion de présenter deux des premières applications des aciers spéciaux. Entretemps, cette technique s'est largement répandue et il nous a paru utile d'exposer brièvement les réflexions faites dans le cadre des études et les conclusions principales découlant des expériences acquises lors de la fabrication et du montage des ouvrages.

2. Caractéristiques des aciers à haute résistance utilisés

Comme l'illustre (Fig. 1) la répartition des matières du pont sur la Veveyse, avec une travée latérale de 111 m imposée par les conditions géotechniques et une travée centrale atteignant 129 m, c'est uniquement le tronçon sur la pile située à la jonction des deux portées précitées, tronçon hautement sollicité à la flexion et au cisaillement, qui est prévu en acier à haute résistance.

Cette région est de plus caractérisée par un rapport assez faible entre les contraintes dues aux surcharges, pratiquement déterminantes pour la fatigue, et celles dues au poids mort et aux charges permanentes. On sait en effet [2] que la résistance à la

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1) Le sigle E 36 désigne un acier à 36 kg/mm^2 de limite élastique, avec une résistance à la rupture d'au moins 52 kg/mm^2. Cette notation est utilisée par exemple dans les recommandations allemandes "Stahl-Eisen-Werkstoffblatt 089 - 70; Schweissbare Feinkornbaustähle; Gütevorschriften" ainsi que "DASt-Richtlinie 011; Anwendung der hochfesten schweissgeeigneten Feinkornbaustähle St E 47 und St E 70 für Stahlbauten mit vorwiegend ruhender Belastung (Ausgabe 1.74)"

fatigue des constructions soudées dépend principalement de la variation des contraintes $\Delta\sigma$ et que celle des éléments en acier à haute résistance est en première approximation la même que pour l'acier doux. Les micro-fissures inévitables ainsi que les effets d'entaille même modérés empêchent en effet de réaliser le gain de résistance que montrent les essais d'endurance sur barreaux polis. Dans les régions à valeurs $\Delta\sigma$ faibles, cette caractéristique en soi défavorable des aciers à haute résistance ne pose pas de problèmes, la limite élastique - et non la résistance à la fatigue - étant le facteur déterminant pour le dimensionnement. Dans les ponts-rails par contre, où la fatigue joue le premier rôle même dans la région des piles, l'acier à haute résistance n'offrira en général pas d'avantages par rapport à l'acier E 36.

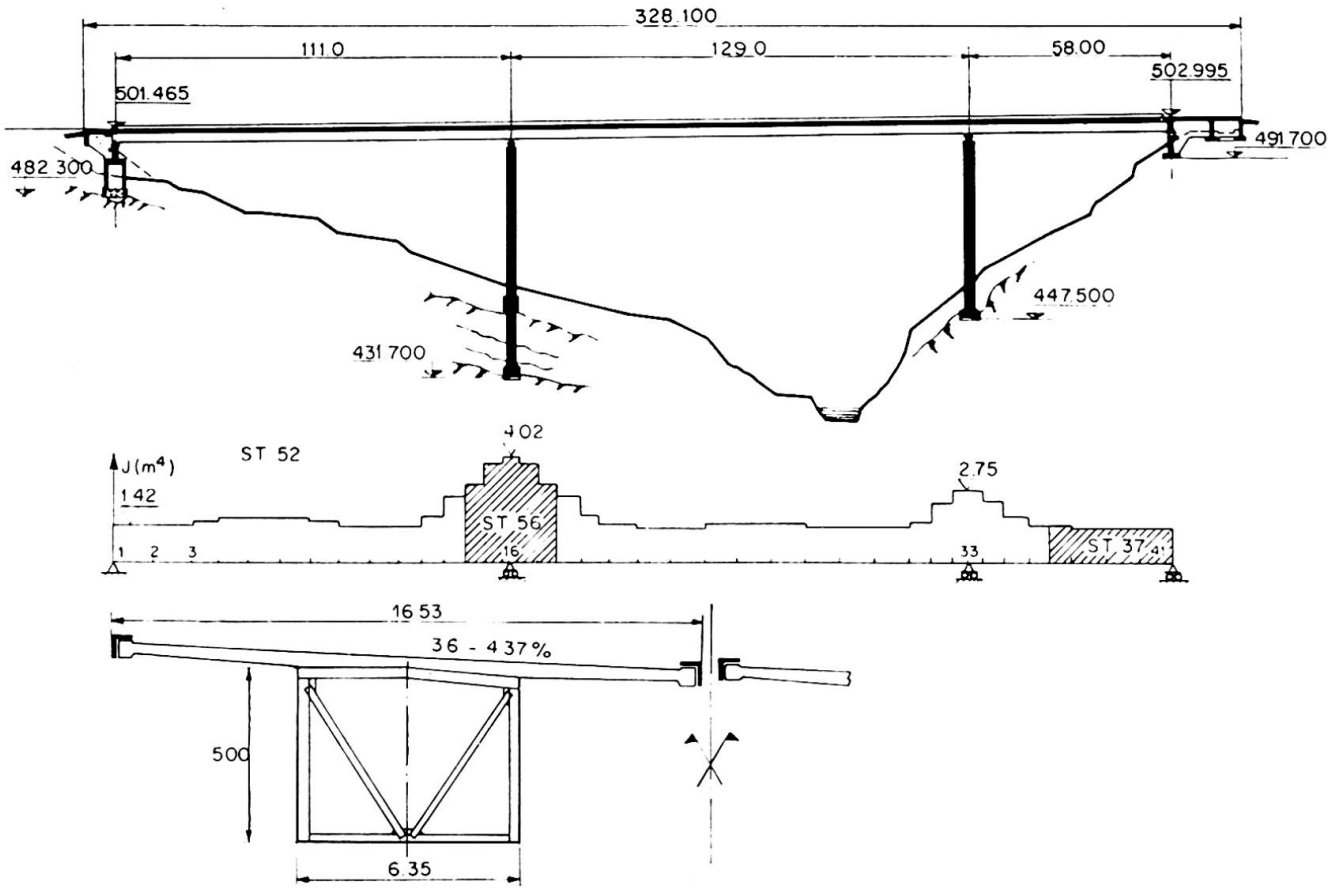


FIGURE 1
Ensemble et section de principe du pont sur la Veveyse

Les aciers à haute résistance utilisés dans les ponts-routes suisses sont généralement du type à résistance naturelle, obtenue comme l'indique le tableau I par des éléments d'alliage appropriés, spécialement des liaisons azotées de nickel ou de vanadium sous forme de dispersoïdes bloquant les glissements cristallins. Ces aciers sont livrés à l'état normalisé grains fins et possèdent une excellente ductilité, contrôlée tant par des essais de résilience que par des pliages Kommerell pour les fortes épaisseurs.

TABLEAU I
Acier à haute résistance E 43 utilisé au viaduc de Bergbach (SG)

Aciérie: AG der Dillinger Hüttenwerke, nuance Dillinal 55/43

COMPOSITION CHIMIQUE (analyse des coulées, en %)

C	Si	Mn	P	S	Ni	V
≤ 0,18	0,10÷0,50	1,0÷1,7	≤ 0,035	≤ 0,035	≤ 0,7	0,10÷0,18

CARACTERISTIQUES MECANIQUES

Limite élastique garantie	≥ 43 kg/mm^2 (≤ 16 mm)
Limite élastique effective	43,6 ÷ 51,7 (t = 16 mm)
(réception en aciérie)	40,7 ÷ 49,2 (t = 62 mm)
Résilience ISO-V, état de livraison (en long)	≥ 5 kg·m/cm^2 à -20° C

La mise en oeuvre de ces aciers en atelier requiert bien entendu les précautions d'usage pour les matériaux à carbone équivalent élevé, en particulier un préchauffage à 80° ÷ 200°, selon l'épaisseur, pour éviter la trempe que provoquerait un refroidissement trop rapide après le soudage ou même après l'oxycoupage. Les procédés opératoires et l'ampleur des contrôles doivent de plus être adaptés à la nuance d'acier choisie, généralement E 43 ou même E 47 à l'heure actuelle, ainsi qu'aux épaisseurs à mettre en oeuvre.

Nous ne saurions ici entrer dans les détails des spécifications adoptées: celles-ci dépendent en partie de l'équipement des ateliers et de l'appareillage de soudage utilisé. On indiquera cependant que les difficultés de fabrication sont légèrement supérieures à celles des ouvrages en acier classique E 36 mais bien inférieures à celles rencontrées pour la mise en oeuvre des aciers trempés et revenus du genre E 70, souvent utilisés en chaudronnerie lourde.

3. Avantages et inconvénients des aciers à haute résistance

On peut se demander quelles raisons parlent en faveur des aciers à haute résistance mentionnés. Nous pouvons éliminer immédiatement le gain de poids puisqu'il s'agit, d'une part, de portées modérées et, d'autre part, des tronçons sur piles dont le poids passe pratiquement directement dans les appuis. Des comparaisons faites sur des ouvrages réalisés ou simplement projetés montrent de plus que le tonnage d'un pont mixte dépend avant tout des choix constructifs et en particulier de l'espacement transversal des poutres[2]; le poids propre de l'ossa-

2) Pour des ponts-routes mixtes comportant deux poutres sous une chaussée large de 15 m au maximum, l'auteur a proposé [3] la formule linéaire $g = 10 + 2\,\ell_m$ pour une première approximation du kilotage d'acier g par m^2 de chaussée en fonction de la portée moyenne $\ell_m = \Sigma\ell_i^2/\Sigma\ell_i$ d'une poutre continue. Une extension de cette relation à des ponts mixtes très larges, mais ne possédant toujours que deux poutres maîtresses a montré [4] que l'on pouvait réduire à 1,5 le multiplicateur de ℓ_m.

ture est de toutes façons faible par rapport à celui de la dalle de couverture et de son tapis bitumineux, de sorte qu'une réduction du poids mort reste pratiquement sans influence sur les sollicitations totales.

L'économie sur le prix des matériaux n'est pas déterminante non plus: le prix relatif des aciers à haute résistance considérés, E 39 ou E 43, c'est-à-dire le prix unitaire rapporté à la limite élastique avec une valeur prise arbitrairement à 100 % pour l'acier doux, est en effet pratiquement égal à celui de l'acier classique E 36 (Fig. 2). Seul l'acier E 70 serait plus avantageux de ce point de vue; il requiert toutefois, comme indiqué plus haut, des précautions coûteuses lors de la fabrication.

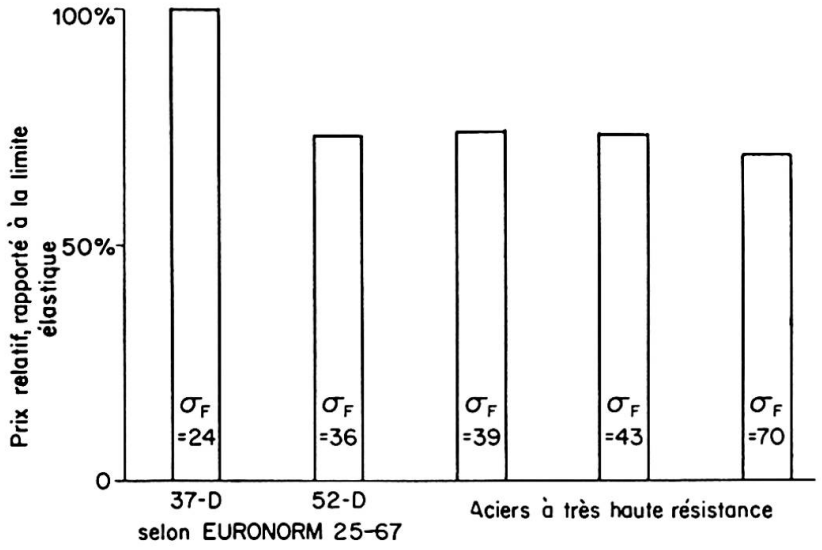


FIGURE 2 - Comparaison du prix relatif de divers aciers

La limite élastique ou, si l'on préfère, les contraintes admissibles sont plus élevées et permettent dès lors de réduire la section des semelles, par exemple de 20 % lorsque l'on passe de l'acier E 36 à E 43. Même dans les régions très sollicitées, une semelle unique est ainsi possible (Fig. 3), de largeur et d'épaisseur permettant une fabrication à la fois aisée et économique (réduction du nombre de cordons longitudinaux) sans poser de problèmes de stabilité en service. L'épaisseur des âmes sera réduite en proportion, ceci au moins dans la région des piles où se limite, comme indiqué précédemment, l'emploi d'aciers à haute résistance; dans cette zone, en effet, l'âme doit être assez épaisse pour reprendre les efforts tranchants très élevés et la stabilité au voilement n'est pas déterminante comme cela serait le cas en travée.

Pour une poutre mixte, la réduction de l'aire des sections diminue de plus les tractions parasites dans le béton dues au retrait empêché. De même, lorsque la dalle est mise en précontrainte par dénivellation d'appuis ou par des câbles pour diminuer le risque de fissuration sur piles, les pertes de précompression seront plus faibles, c'est-à-dire que le rendement sera plus élevé.

On ne dispose pas encore en Europe d'aciers patinables à haute résistance, c'est-à-dire avec des limites élastiques supérieures à

celle de l'acier E 36. Ce fait est regrettable puisque les aciers patinables offrent des avantages certains dans l'entretien des ponts. Espérons que les progrès de la métallurgie permettront sous peu de remédier à cet inconvénient qui risquerait de freiner l'extension de l'utilisation des aciers à haute résistance.

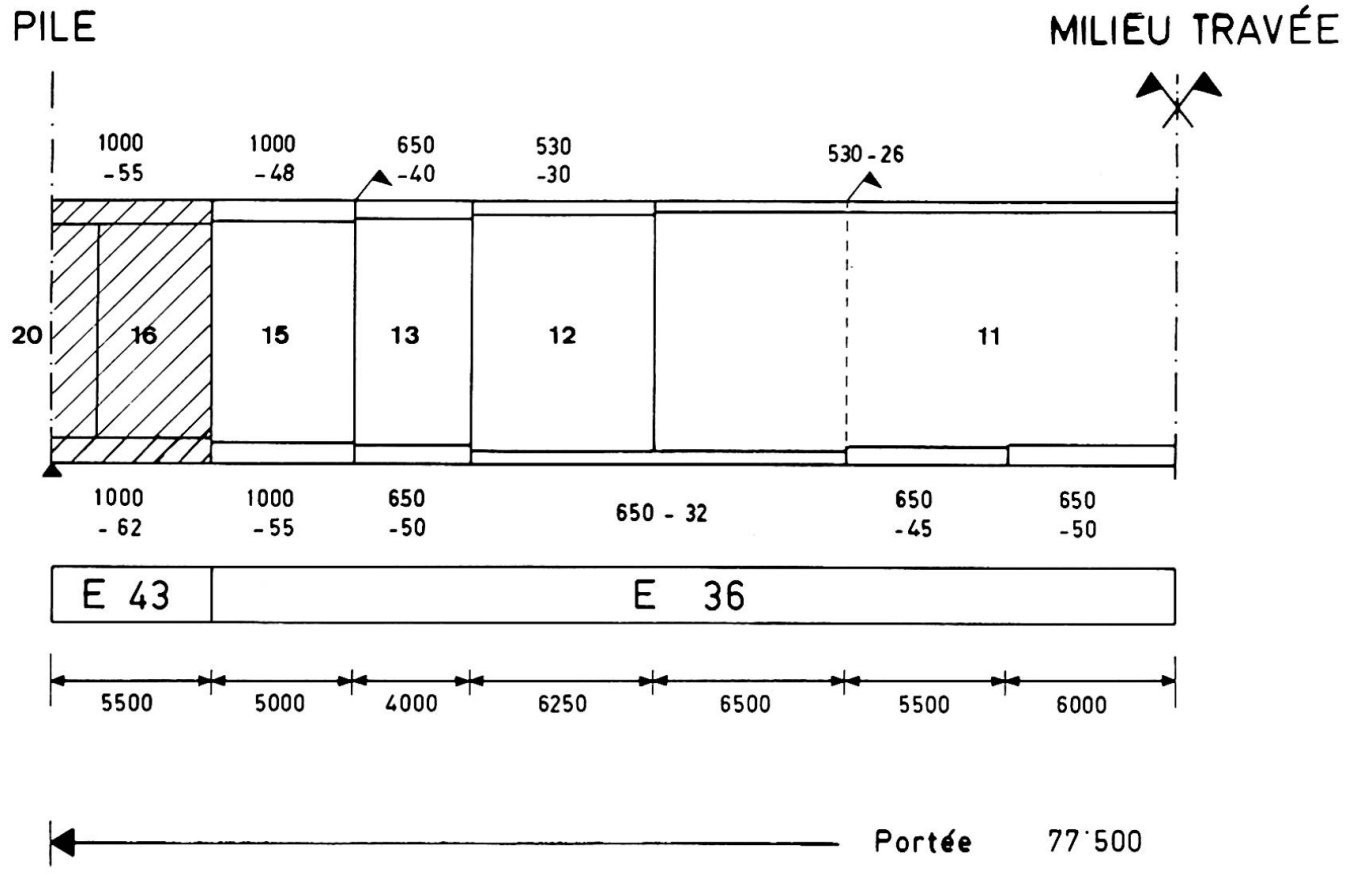


FIGURE 3 - Viaduc de Bergbach (SG): répartition des matières dans les travées centrales

4. Particularités de la mise en oeuvre

On mentionnera pour conclure certaines particularités de la mise en oeuvre en atelier et au montage. L'utilisation de l'acier à haute résistance étant réservée aux tronçons sur piles, les semelles correspondantes ne comportent généralement pas de joints intermédiaires et les cordons de soudure transversaux à leurs extrémités concernent des épaisseurs déjà plus faibles (voir Fig. 3 plus haut).

Les ponts dont nous nous occupons ont été montés par lancement; l'assemblage se fait dès lors au sol dans des conditions favorables. On soudera même durant l'hiver (Fig. 4) en prenant les précautions d'usage comme cabines de protection, etc. Pour l'ouvrage visible sur la figure, on a utilisé le soudage sous CO_2 avec fil fourré de flux aggloméré basique.

Le lancement sur la travée latérale de 111 m (Fig. 5) a tout au moins prouvé que les soudures présentent une excellente résistance à des sollicitations statiques très élevées!

FIGURE 4
Pont sur la Veveyse. Assemblage au sol avant lancement.

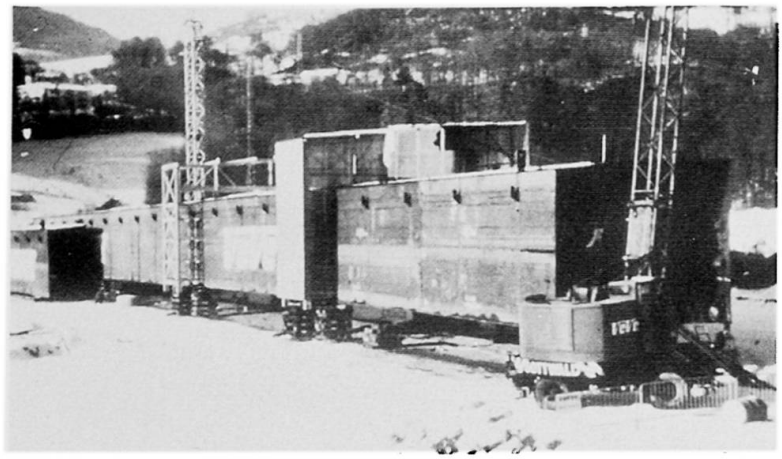
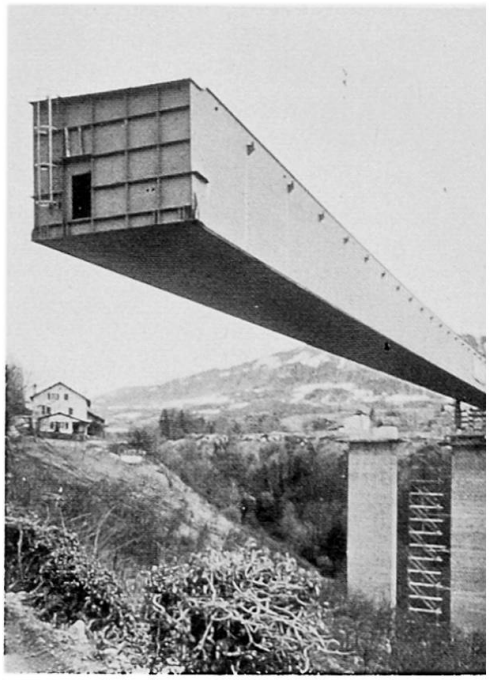


FIGURE 5 →
Pont sur la Veveyse. Ouvrage à la fin du lancement; rayon de courbure en plan 907 m.



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RESUME

On présente quelques exemples d'utilisation d'aciers à haute résistance dans la région des piles de ponts-routes mixtes continus. On expose les avantages résultant de l'emploi de ces aciers et les particularités de mise en oeuvre.

ZUSAMMENFASSUNG

Es werden einige Beispiele der Verwendung hochfester Stähle im Stützenbereich durchlaufender Verbundstrassenbrücken dargestellt. Neben einer Beschreibung der Besonderheiten der Verarbeitungsverfahren erwähnt man auch die Vorteile, die sich aus der Anwendung dieser Stähle ergeben.

SUMMARY

Some examples are given of the use of high strength steels in the support regions of continuous composite highway bridges. The advantages resulting from the use of those steels are outlined and the particularities of the fabrication methods described.

Steel Bridge Decks Realized with Corrugated Plates and Plane Sheets Connected by High Strength Bolts

Tabliers de pont métallique réalisés au moyen de tôles pliées et planes reliées par des boulons à haute résistance

Stählerne Fahrbahntafeln aus mit hochfesten Schrauben verbundenen kaltgeformten und ebenen Blechen

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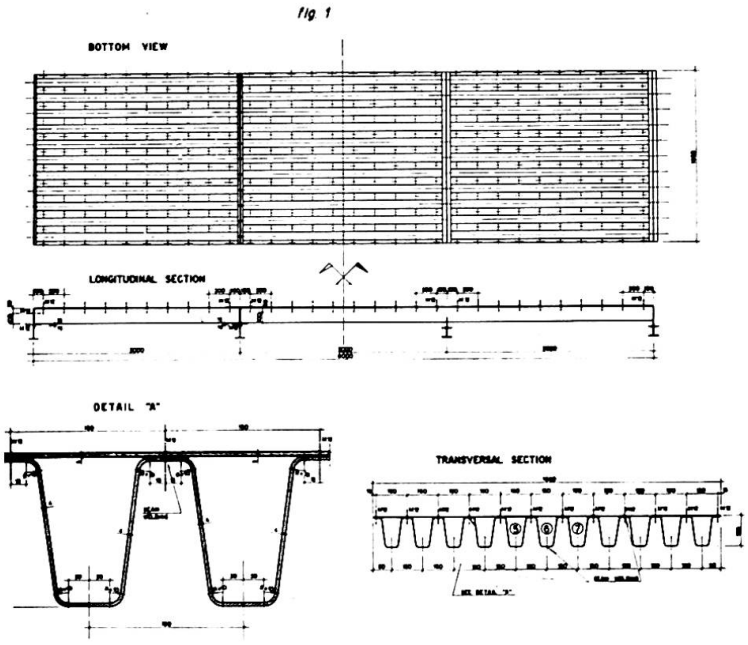
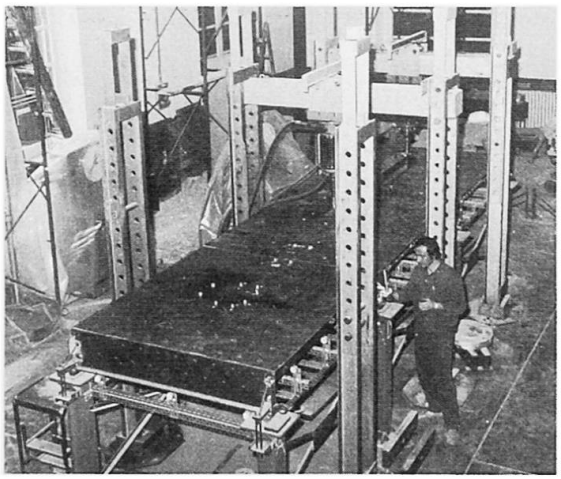
Introduction

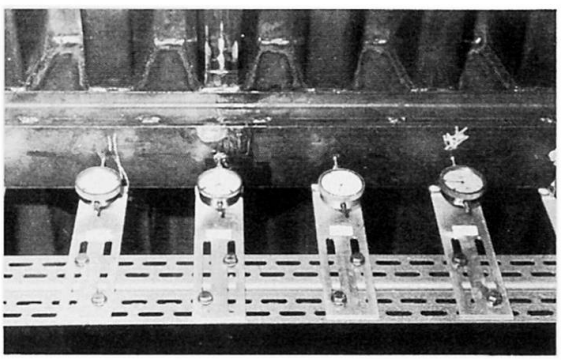
The results of an experimental research on a model of steel orthotropic bridge deck are here briefly summarized.

Tests were performed in the University of Rome in collaboration with the Italsider society during the years 1973-75.

The model, scale 1:2, was made by an upper plane sheet connected by high strength bolts to a botton folded sheet, the resulting deck being similar to an orthotropic plate with closed ribs (fig. 1 2 and 3).







In such a type of deck weldings are almost completely eliminated, with reduction of costs expecially when highyield strength steels are employed.

This type of deck could be adopted also in civil or industrial buildings where heavy liwe loads must be carried with limitation on permanent loads.

Tests in the elastic range

Static tests in the elastic range have been performed by loading the plate in a number of points with two types of load. One acting on a small area simulated the action of a truck wheel; load acting on a large area represented the action of coupled wheels.

By making use of mechanical devices, the absolute displacements of the plate and tre relative displacement between bolts and sheet have been meastured. Strains have been measured by means of electrical strain gages.

From the several tests performed the following conclusions can be drawn:

a) the connection of flat sheet and corrugated sheet by means of high strength bolts is reliable due to the fact thet relative displacements have not been noticed and the behaviour exhibited by the model has been nearly linear.

b) the torsional rigidity of this deck type is small and similar to the one of orthotropic plates with open ribs. Fig. 4 compares the values of stresses meaured at midspan of ribs and the corresponding theoretical values furnished by the Pelikan-Esslinger method for several values of the relative stiffness. A good agreement is achieved for values of x as little as 0.2. This effect is seemingly due to the shape loss of the bottom corrugated sheet.

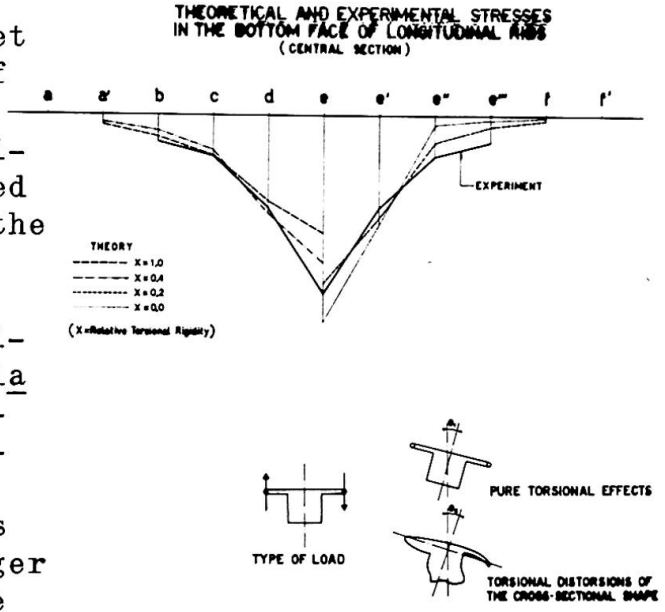


fig. 4

c) local stresses within the loaded area are rather high due to the fact that the deflection length of the top flat sheet is of the order of the bolts distance (fig.5).

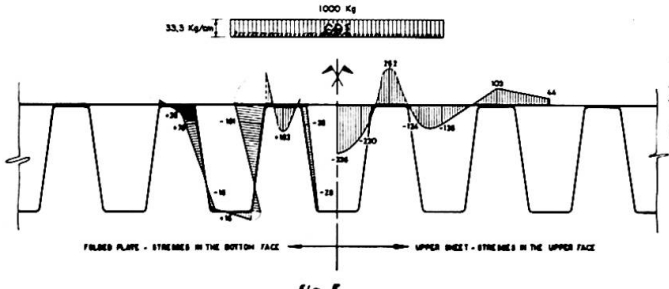


fig. 5

With regard to points b) and c) a better behaviour of the model could be obtained with a double row bolts, possibly in a "quinconces" setting.

Fatigue tests

Tests under pulsating loads have been performed to get information on the predominant types of fractures and on the importance of fatigue in comparison with the static behaviour.

The minimum value of the pulsating load was about 1/20 of the maximum and therefore it can be assumed, for all practical purposes, as equal to zero.

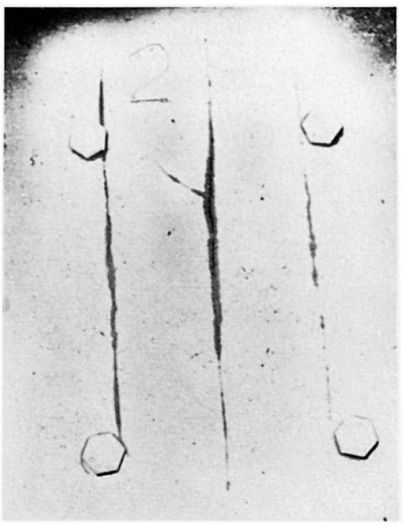
The load has been applied at seven different spots and it has been increased after a suitable number of cycles.

Displacement have been recorded by means of trasducers. Penetrating liquid of the Spotcheck type has been employed in order to spot fractures.

The result of tests are here summarized:

a) local fractures occur in the top flat plate within the loaded area and are of the type shown in fig. 6.

b) stresses which cause fracture are nearly the same as those associated with fatigue fracture in a flat sheet under pure bending, i.e. no reduction in strength has been noticed because of the presence of bolts.



Collapse test

A section of the model has been tested under proportional loading up to collapse after the fatigue fractures have been repaired.

The load has been increased stepwise, each step being 1/10 of the theoretical collapse load. At each level a numer of loading cycles has been performed until a steady situation has been reached (differences between deformations of two subsequent cycles smaller than 5/100 mm).

In the central loaded area stress coat paint has been used to spot yielded regions.

The test confirmed that the model presents a very high load-carrying capacity, collapse load being about ten times the service load. This is due to a progressive load transfer from the loaded area to the adjacent areas caused by the membrane behaviour of the top flat sheet. This effect can be clearly seen in fig. 7 where plots of loads vs displacements are shown, as well as in fig. 8 which shows stresscoat paint fracture on the top sheet near the load.

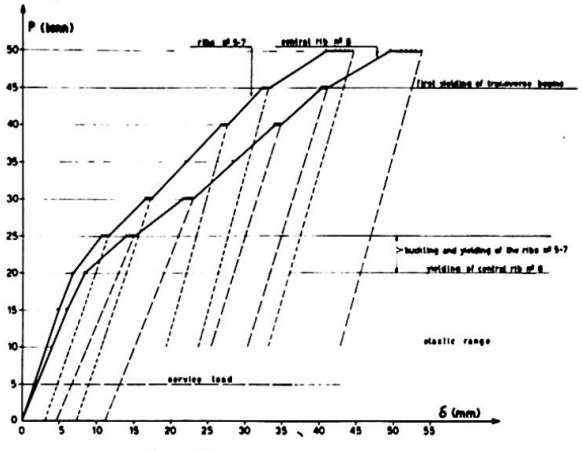
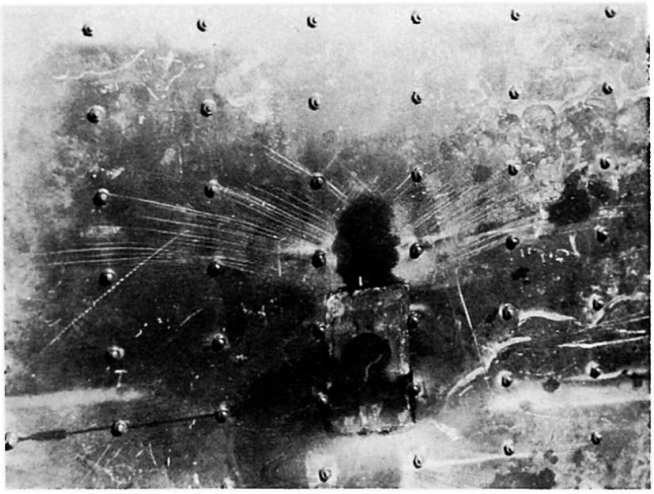


fig. 7



The continuation of the research with forces acting in the plane of the plate is in future scheduled. These forces will probably emphasize local instability with subsequent reduction of the load-carrying capacity of the model.

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SUMMARY

The results of an experimental research on a model of steel bridge deck are briefly summarized. The model was made by an upper plane sheet connected by high strength bolts to a bottom folded sheet to form a deck similar to an orthotropic plate with closed ribs. Static tests in the elastic range as well as tests under pulsating loads and under proportional loading up to collapse have been performed.

RESUME

On présente les résultats d'une recherche expérimentale sur un modèle de tablier de pont métallique. Le modèle était constitué par une tôle supérieure plane reliée au moyen de boulons à haute résistance à une tôle pliée inférieure de façon à former un tablier du type plaque orthotrope avec nervures fermées. On a effectué des essais statiques dans le domaine élastique, des essais sous charges variables et enfin des essais de chargement proportionnel jusqu'à la rupture.

ZUSAMMENFASSUNG

Die Ergebnisse von Versuchen an einem Modell einer stählernen Leichtfahrbahntafel werden kurz erörtet. Das Modell wurde mit einem oberen ebenen Blech hergestellt, das durch hochfeste Schrauben mit einem unteren kaltgeformten Blech verbunden war, so dass eine Art orthotroper Platte mit Hohllängsrippen entstand. Die Versuche wurden statisch im elastischen Bereich, auf Ermüdung mit Ursprungsbelastung sowie bei Proportionalbelastung bis zur Traglast durchgeführt.

Welding of High-Strength Steels

Soudage des aciers à haute résistance

Schweissen von hochfesten Stählen

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1. INTRODUCTION

It should be taken great pains with not only the characteristics of materials but also the welding methods from a viewpoint of risk of failure of long-span bridge made of high-strength steels. In the preliminary report, the outline of way to build-up of long-span truss bridge, i.e., design, materials, fabrication and erection were discussed. In this report, the conditions and methods of welding and the quality control of welding during the fabrication course may be presented.

2. CONDITIONS OF WELDING

As a procedure test to determine welding conditions of high-strength steels 70 kg/mm^2 and 80 kg/mm^2 classes, the following tests were conducted prior to fabrication;

(1) lamellar tear test,
(2) restrained cracking test,
(3) tests on the performance of corner weld joints,
(4) tests to check residual stresses due to welding, and
(5) tests to investigate the various characteristics of the actual members using full scale models.

The maximum heat input of less than 50 KJoule/cm during welding was preset based on the results of such tests to avoid "bond" embrittlement of weld joint. Welding conditions including minimum preheat temperature, maximum preheat temperature, interpass temperature, and preheat temperature for tack welds were determined from the condition mentioned above. (Table 1)

The three factors of kind of materials, restraining and hydrogen are said to be main ones causing weld cracking. The first was settled by employing the steels passing the new specification. The second was thought not to be serious problem because of no steel plate having great restraining with neighbour plate in the Osaka Port Bridge. Therefore, the final factor was remained as the most considerable one which brought so called transverse cracking, i.e., the cracking along weld line. The suitable control method with which hydrogen entry could be minimized as much as possible was settled for fabrication in order to avoid such possible cracking.

Table 1 Preheat Temperature

Minimum Preheat Temparature (°C)

High-Strength Steels of 70 kg/mm² and 80 kg/mm² Classes				
Welding Method / Thickness (mm) \ Kind of Joint	Covered Arc Welding	M I G ※ Welding	Submerged Arc Welding	
	Butt , Fillet , Corner		Butt ,	Fillet
t ≦ 50	100	80	100	80
t > 50	120	100	150	100

※ M I G Welding.....Inert Gas Shielded Metal Arc Welding.

Maximum Preheat Temparature and Interpass Temparature

Thickness (mm)	Temparature
t ≦ 50	200°C and under
t > 50	230°C and under

3. WELDING METHOD

The automatic welding technique was made every effort to use in order to uniformalize the fabrication conditions. Among the automatic methods, two techniques of submerged arc welding and MIG welding were selected. The latter has an advantage to avoid the possible occurrence of cracking because of small heat input, but involves the serious problems of ill efficiency of work and of increased hardness in weld joint. Therefore, the first was used as the butt and fillet welding and the latter as the corner one. Also the following three types of preheating methods were used;

(1) electric preheating type with automatic control, (Photo 1)
(2) fixed burner type, and
(3) manual burner type.

The separate preheating method was specified for each type of weld joints. In welding major members, symmetrical preheating and symmetrical welding methods were employed to secure higher fabrication accuracy and minimize residual stresses. (Photo 2)

The preheating prior tack welding of small heat input and rapid cooling was carried out with careful concern, and its temperature was set higher by 20-40°C than that of primary weld.

4. QUALITY CONTROL OF WELDING

The close quality control system is necessary for the use of high-strength steels 70 kg/mm^2 and 80 kg/mm^2 classes to long span bridge. Although the detail explanation of such system is omitted in this report, the non-destructive inspections involves serious and many problems. Especially at the ultrasonic examination of corner joint shown in Fig.1, as the ultrasonic flaw detector possessed by the fabricators were different in the kind and the material and dimension of probe, the inspection was quite different each other, and the correlation between flaw and echo from discontinuity captured on the Braun tube could not be obtained. Therefore, the inspection standard was rechecked from the measurements with various test material.

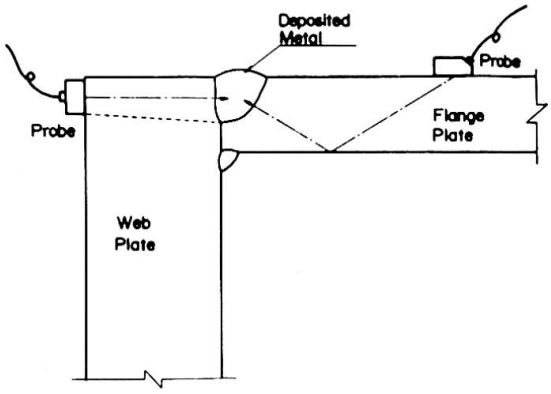

Fig. 1 Corner Joint

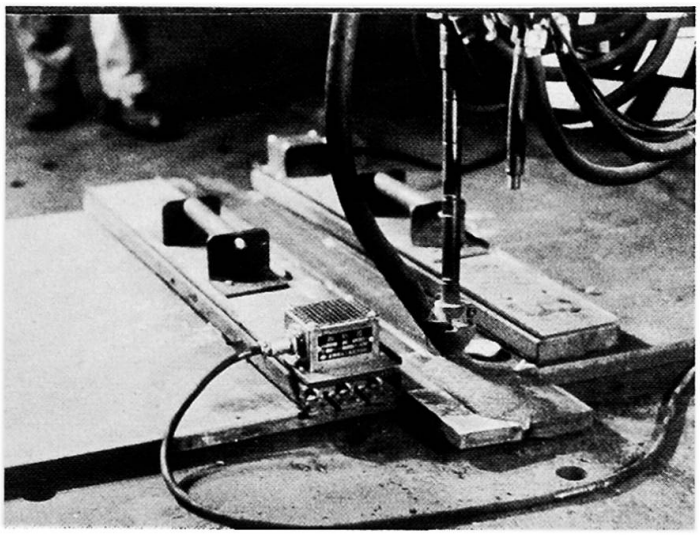

Photo 1

This photograph shows preheat performance with electric preheating equipments.

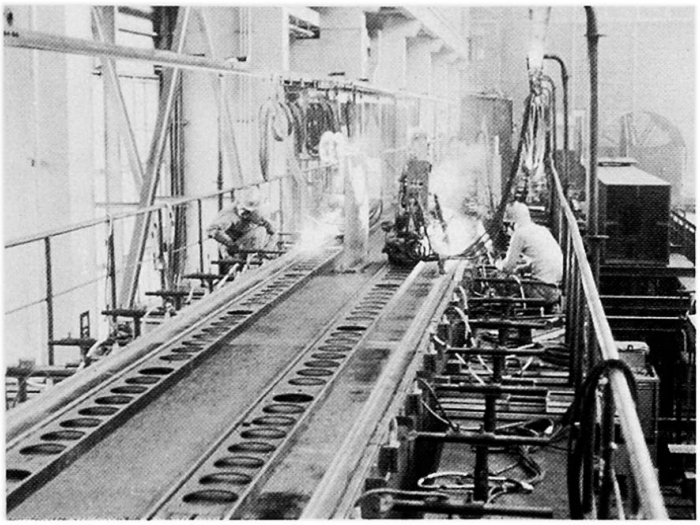

Photo 2

This photograph shows the appearance of symmetrical welding.

SUMMARY

Due attention was paid to the problem of cracking and embrittlement of weld joints of high-strength steels. The maximum heat input of less than 50 K Joule/cm during welding was preset and the preheat temperature was determined. The suitable control method with which hydrogen entry could be minimized was settled for fabrication. Among the automatic welding methods, two techniques of submerged arc welding and MIG welding were selected.

RESUME

On a prêté une attention particulière au problème des fissures et des ruptures fragiles dans les joints soudés en acier à haute résistance. L'apport de chaleur durant le soudage a été limité à 50 K Joule/cm et la température de préchauffage, déterminée. On a fixé la méthode de contrôle permettant de réduire au minimum la teneur en hydrogène. Parmi les méthodes de soudage automatiques, on a retenu le soudage sous flux et le procédé MIG.

ZUSAMMENFASSUNG

Der Vermeidung von Rissen und Sprödbrüchen in Schweissnähten an hochfesten Stählen wurde besondere Beachtung geschenkt. Das Wärmeeinbringen während der Schweissung wurde auf 50 K Joule/cm begrenzt und die Vorwärmetemperatur bestimmt. Man verwendete eine geeignete Kontrollmethode, um die Wasserstoffaufnahme möglichst niedrig zu halten. Unter den automatischen Schweissmethoden wurden das Unterpulverschweissen und das MIG-Verfahren ausgewählt.

V

Synthesis and Conclusions

Synthèse et conclusions

Synthese und Schlussfolgerungen

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Prepared Discussion

The numbers of papers presented at the prepared discussion are seven for theme Va,two for theme Vb and two for theme Vc. The following is a short summary of these contributions.

Theme Va. Starting with theme Va."Structural Behavior Including Hybrid Construction", P.Ferjencik reported on the structural behavior of prestressed steel beams. A wide flange beam is prestressed by means of high-strength tension wires installed just beneath the lower flange. Test results showed that elastic limit capacity of this system increases remarkably compared with that of non-prestressed regular steel beam. Due to the effect of prestressing and of plastic reserve strength,the ultimate strength of this system can attain as high as twice the computed ultimate strength of non-prestressed beam if the premature lateral and local buckling is carefully prevented.

M.Yamada and B.Tsuji carried out an elastic-plastic analysis of hybrid beam-columns subjected to repeated and reversed bending under constant axial force. It was demonstrated that in hybrid beam-columns,the expansion of hysteresis loops for each additional cycle can not be expected so much when the magnitude of the axial force is large.

Z.Cywinski carried out a plastic analysis of hybrid beams on the basis of lower bound criterion,and pointed out that for short span beams,the effect of shear stress becomes more severe than that assumed in the present ASCE-AASHO regulation.

A.Plumier investigated the fatigue strength of plate girders with rather unique details.Main series of test girders had transverse stiffeners both ends of which were welded to upper and lower flanges. A girder in which only one end of stiffener is welded to tension flange was also tested as a supplement. Both hybrid girders and homogeneous girders made entirely of C-class steel were tested. Obtained results are that the fatigue strength of these girders are inferior than that of regular girders in which stiffener is welded at compression flange only, and that there are no difference in fatigue strength between these two types of girders tested and fatigue strength of homogeneous girders is a bit superior than that of hybrid girders. J.Janss has reported this in lieu of Plumier.

Y.Maeda reported an additional study on type 2 cracks in hybrid girders which supplements his previous paper presented in Preliminary Report. He checked the

effect of finishing of weld toes,and showed that fatigue strength will increase about 1 kg/mm^2 by this finishing. However this effect will be covered up by other fluctuating factors such as residual stress,quality of weldment and misalignment of loading system.

Also as a supplement of his investigation presented in the Preliminary Report, T.Yamasaki discussed on the possibility of estimating fatigue lives by means of fracture mechanics approach. He showed that fatigue lives can be predicted by using material constants obtained from a simple test,and thus the fracture mechanics approach is useful for fatigue problem also,if the size of initial crack could be estimated in advance.

The COD criterion is widely used in the field of non-linear fracture mechanics However,when the yield region is spreaded widely enough,the physical meaning of COD theory becomes not so clear compared with the concept of J_{1c} fracture criterion. Furthermore,since the J_{1c} value is rather insensitive to the scale effect,one can determine the J_{1c} value using the smaller test specimens. H.Aoki had evaluated J_{1c} values for B and C-class steels,and showed that J_{1c} value of C-class steel is about 2/3 of that of B-class steel. He also pointed out that critical J-value will decrease with the increase of multi-axiality of the stress field.

Theme Vb. First discussion on theme Vb "Design Problems" was contributed by J.Lindner. He discussed on the lateral buckling strength of beams with mono-symmetric cross sections. It is well known that a mono-symmetric cross section of which the upper flange is larger than the lower flange is more advantageous than a doubly symmetric section of equal sectional area as far as elastic bifurcation is concerned. In plastic region,however,he pointed out that the buckling strength of a beam having mono-symmetric cross section will become lower than that having doubly symmetric cross section,and the merit of high-strength steel becomes very small.

P.Dubas talked about the utilization of high-strength steels in the design of composite girder highway bridges in Switzerland refering to the Veveyse bridge and others designed by himself. High strength steel currently used in Switzerland is almost limited to B-class steel. He remarked that though the weight saving and the reduction of material cost expected from the use of high-strength steel is not substantial, some merit can be obtained by performing a good design. A merit will be brought by using high-strength steel at the end portions of each span of a continuous girder where negative moment and shear force are very high. Sectional area at this portion will decrease by this selection and thus the almost uniform cross section will be obtained throughout the span which makes the fabrication very simple. Since the shrinkage of concrete is restrained by shear connectors,tensile stress will arise in the concrete slab. This tensile stress will be reduced by the use of high-strength steel since the sectional area and thus the rigidity of the steel beam is reduced. This is another advantage. He also encouraged the development of the weathering high-strength steel which will offer a merit with respect to the maintenance.

Theme Vc. In theme Vc "Fabrication and Election Problems", a new type of steel orthotropic bridge deck was introduced by M.P.Petrangeli. The upper plane sheet and corrugated metal deck were connected by means of high strength bolts. This fabrication method was adopted by considering the fact that the welding becomes very difficult and expensive when these decks are made of high-strength steel. The structural performance of this deck was investigated by model tests. Elastic behavior,ultimate strength under static loading and fatigue strength were investigated to obtain the satisfactory results.

As a supplement to his contribution to the Preliminary Report,M.Sasado discussed on the welding procedure of high-strength steels used in the construction of Osaka-port bridge. C-class steels up to 75mm in thickness were used in this bridge. The condition of welding,welding methods and quality controll adopted in this construction to avoid the possible cracking and embrittlement at welded joints were reported.

Free Discussion

Nine topics were contributed in the form of free discussion; they are three for theme Va,five for theme Vb and one for theme Vc.

Theme Va. As the results of additional tests on composite beams with formed steel deck undertaken after the publication of their paper in the Preliminary Report, I.M.Viest and J.W.Fisher proposed some modifications of eq.(2) and eq.(4) presented in the Preliminary Report as follows;

$$Q_{rib} = \frac{0.85}{\sqrt{n}} \left(\frac{H-h}{h}\right)\left(\frac{w}{h}\right) Q_{sol.} \leq Q_{sol.} \qquad (2a)$$

where n=number of shear connectors placed in a rib.

$$I_{eff.} = I_s + \sqrt{\frac{V'_h}{V_h}} \left(I_{tr} - I_s \right) \qquad (4a)$$

O.Jungbluth conducted a series of load carrying tests on portal frames made of various grades of steel,and demonstrated that the simple plastic theory may be applicable even for the rigid frame made of C-class high-strength steel (σ_y=685 N/mm^2).

A.C.Wallace discussed on the structural behavior of connection details of the tied bowstring arch bridge made of high-strength steel refering to an actual example.

Theme Vb. N.Streletzky talked about the application of high-strength steels to building and bridge structures in UdSSR. High-strength steels newly developed in UdSSR are 16Г2Aф(σ_y=440 N/mm^2, C'-class) and 14Х2ГМР(σ_y=590 N/mm^2,C-class). Brief comments on the allowable stress and the method of analysis were also given.

The effect of exposure on corrosion of weathering high-strength steel was discussed by W.I.J.Price. He examined the difference between the formation of the protective coating achieved in open exposure and that obtained in a sheltered environment which is the case of composite bridge where the concrete deck shelters the steel beam from direct rain and sun. He demonstrated that the sheltered corrosion rate is higher for marine and saline situations,and also pointed out that severe pitting observed in these environments will cause the adverse effect against fatigue and brittle fracture.

W.Hoyer evaluated the limit of adaptability of weathering high-strength steel for various environmental and service conditions based on a extensive research work.

The reduced modulus of a stayed cable was discussed in both the Introductory Report Vb and the Preliminary Report by Hajdin both of which are based on the formula proposed by Ernst in 1965. M.Ito derived a revised formula which is theoretically more reasonable. It was revealed,however,that Ernst's formula could give practically satisfactory results except for the case where a very long cable is subject to relatively low tensile stress.

K.Horikawa calculated the stress intensity factor which may arise in a welded joint of C-class steel(HW80) containing a possible crack which might be overlooked through the regular nondestructive inspection,and pointed out that the critical toughness of such a joint material must be larger than that calculated value.

Theme Vc. As one of the means to obtain column-to-beam rigid connections,high strength bolted T-stub flange-to-column connections are widely used. K.Sato talked about the utilization of high-strength cast steel T-stubs for this connection system. When beam ends are subjected to very large bending moment due to earthquake or wind force,these attachments can be utilized effectively.

Appendix.

The list of high-strength steels attached at the end of theme Va. of the Introductory Report should be supplemented by the following two high-strength steels. One is of United States,and this steel has been already introduced at the beginning of this session by the author. The other is of F.R.G.(West Germany) which is quite similar to A514 steel of United States,and to HW 70 steel of Japan in quality. This steel was used by J.Lindner and by A.Plumier in their research and reported in this prepared discussion. With respect to 16Г2Aφ and 14X2ГМР steels of UdSSR refered by Streletzky,detailed information is not available.

New High-strength Steels to be Added to Appendix of Introductory Report

Country	Standards	Designation	Min.Yield Stress N/mm^2	Tens. Strength N/mm^2	Class	Y
U.S.A	ASTM	A572,Gr 65	448	552	C'	0.812
F.R.G (West Germany)	DASt. Ri.011	St.E 70	686	785 ~ 932	C	0.875

$Y = \frac{\text{Yield Stress}}{\text{Tensile strength}}$ = Yield ratio of material(maximum)

VI

Constructions en béton préfabriqué

Vorfabrizierte Bauwerke

Precast Structures

VI a

Sécurité et stabilité des éléments et des constructions

Sicherheit und Stabilität von Elementen und Bauwerken

Safety and Stability of Elements and Structures

VI b

Développements dans la production et l'assemblage

Entwicklungen in Herstellung und Montage

Developments in Manufacture and Assembly

VI c

Utilisations nouvelles, comprenant les constructions sous-marines et flottantes

Neue Anwendungen einschliesslich Unterwasserbauten und schwimmende Konstruktionen

New Applications including submerged and floating Structures

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Bemerkungen des Verfassers des Einführungsberichtes

Comments by the Author of the Introductory Report

Remarques de l'auteur du rapport introductif

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Sicherheit und Stabilität von Elementen und Bauwerken

Die Fragen nach der Sicherheit und Stabilität von Baukonstruktionen aus vorgefertigten Traggliedern werden hauptsächlich durch die Eigenschaften der Verbindungen zwischen den Fertigteilen beantwortet. Es ist deshalb logisch, wenn die Beiträge des Vorberichtes zum Thema VIa besonders diesen Aspekt behandeln.

Bemerkenswert ist weiter, daß sich diese Beiträge fast ausschließlich mit Hochbauten der Plattenbauweise befassen. Die Ursache dafür liegt in der stark steigenden industriellen Produktion von mehrstöckigen Wohnhäusern und ähnlichen funktionell Wabenzellen bedingenden Hochbauten.

Das Verhalten vorgefertigter Bauwerke unter dem Einfluß horizontaler Kräfte, die aus Wind und insbesondere infolge Erdbeben auftreten wird von V a r s a n o theoretisch und von dem Autorenkollektiv um O k a m o t o mit einer großartig durchgeführten experimentellen Untersuchung beleuchtet. Die theoretische Untersuchung führt auf das Problem der Ermittlung der Eigenschwingzeit derartiger Fertigteilkonstruktionen. Infolge der dabei zu treffenden noch nicht ausreichend erforschten Annahmen können die bisher erzielten Ergebnisse nur als eine grobe Näherung betrachtet werden.

Aus einem Vergleich der Eigenschwingzeit, der mit den empirischen Formeln der französischen und amerikanischen Vorschriften zwischen vorgefertigten Plattenkonstruktionen und monolithischen Skelettkonstruktionen von V a r s a n o durchgeführt wurde, leitet er die These ab, daß die horizontalen Kräfte bei vorgefertigten Plattenkonstruktionen größer als bei monolithischen seien. Diese These kann man jedoch nicht ohne weiteres verallgemeinern,denn nicht alle vorgefertigten Platten- und Skelettkonstruktionen sind steifer und massemäßig so unterschiedlich zu monolithischen Konstruktionen, daß dadurch bedingt größere Horizontalkräfte auftreten müssen. Dies zeigen auch die von O k a m o t o u.a. beschriebenen Versuche an originalgroßen Gebäude-

sektionen. Durch zweckmäßig ausgebildete Verbindungen zwischen den Fertigteilen kann das Verhalten von vorgefertigten Bauwerken so entscheidend beeinflußt werden, daß sie eine "ausreichende Tragfähigkeit und genügendes Verformungsverhalten aufweisen, um auch schweren Erdbeben zu widerstehen".

Leider wurden die mitgeteilten Versuchsergebnisse von O k a m o t o u.a. im Beitrag noch nicht mit theoretischen Analysen verglichen. Es ist jedoch bemerkenswert, daß die von O k a m o t o u.a. von zwei Versuchsobjekten mitgeteilten Eigenschwingzeiten recht gut mit Werten übereinstimmen, die sich ergeben, wenn man die von V a r s a n o zitierte Formel (4) der französischen Vorschriften benutzt. Das Versuchsobjekt 1 ergab eine Eigenschwingzeit von 0,1 s und die empirische Formel liefert dafür den Wert 0,125 s. Für das Versuchsobjekt 3-1 wurden 0,12 s ermittelt und die genannte Formel ergibt 0,106 s. Mit anderen Worten, durch die japanischen Versuche konnte die in den französischen Vorschriften von 1969 festgelegte Formel für die Berechnung der Eigenschwingzeit von Fertigteilhäusern für diese Abmessungen untermauert werden.

Bekanntlich sind Explosionen durch Gas oder Heißwasserbehälter in Wohnhäusern trotz umfangreicher Sicherheitsmaßnahmen nicht völlig auszuschließen, wie dies u.a. auch das von V a r s a n o mitgeteilte Beispiel einer Heißwasserboiler-Explosion im 4. Geschoß eines 8-stöckigen Gebäudes in Jerusalem im Juli 1975 beweist. Es ist deshalb notwendig, diesen Lastfall beim Entwurf von Fertigteilhäusern so zu berücksichtigen, daß beim Versagen eines tragenden Bauteils kein fortschreitender Einsturz auftreten darf. Diese notwendige Forderung mit ökonomisch vertretbarem Aufwand zu realisieren, war und ist das Ziel vieler Forschungsarbeiten.

V a r s a n o unterbreitet in seinem Beitrag den konstruktiven Vorschlag, ein dreidimensionales Bandsystem einzubauen, das im Versagensfall einer oder mehrerer Platten die Kräfte zur Stabilisierung des gesamten Bauwerkes aufnehmen kann. Seine dazu gemachten konkreten Angaben über die Ausbildung der vertikalen Fugen, der Schnittpunkte und der Deckenscheiben mit Ringanker werden teilweise bereits bei einer Reihe von Bauwerken, so in der DDR, der UdSSR und anderen Ländern mit Erfolg angewendet.

Das Autorenkollektiv H a n s o n u.a. informiert hierzu über Versuche an 3/8-Modellen. Dabei wurde das Kragverhalten durch Ausfall einer tragenden Wand im unteren Geschoß untersucht. Für die durch senkrechte Zugglieder und waagerechte Spannglieder sehr wirkungsvoll hergestellten Verbindungen der Wand- und Deckenplatten, wie sie für Bausysteme in Nordamerika üblich sind, konnte ein ausreichendes Kragverhalten für die über der ausgefallenen Wand liegenden Geschosse bei Eigenlast plus 0,33 der Verkehrslast, die als Normalbelastung solcher Gebäude betrachtet werden kann, festgestellt werden. Weitere Versuche sollen zu konkreten Festlegungen für die Berechnung und Ausführung von Plattenfertigteilhäusern unter Beachtung solch außergewöhnlicher Lastfälle führen.

In diesem Zusammenhang möchte ich auch auf den Beitrag im Vorbericht von H e y n i s c h zum Thema Ib hinweisen, wo zum Problem der Gasexplosion im Wohnungsbau Ergebnisse über theoretische und experimentelle Untersuchungen mitgeteilt werden, die u.a. in einem Diagramm münden, aus dem der Belastungsimpuls in Abhängigkeit vom Volumen des Explosionsraumes und der Entlastungsfläche abgegriffen werden kann. Aus diesen Untersuchungen leiten sich auch die Hinweise für den Entwurf wie

- Anordnung großer Entlastungsflächen
- Gasinstallation nur in einem Raum je Wohnung
- keine langgestreckten oder abgewinkelten Explosionsräume

ab.

Wenn auch noch weitere ökonomische, insbesondere technologische und konstruktive Verbesserungen zur Vermeidung von progressiven Einstürzen ganzer Bauwerkssektionen oder Bauwerke der Plattenbauweise möglich und notwendig sind, kann doch zunächst festgestellt werden, daß mit den zwischenzeitlich praktisch erprobten Lösungen das Problem für den außergewöhnlichen Lastfall der Explosion in Wohnungen einigermaßen beherrscht wird.

Auf das Problem des Ausknickens und die verminderte Tragfähigkeit infolge exzentrischer Beanspruchung der schlanken tragenden Wandplatten weist V a r s a n o besonders hin. Er verdeutlicht durch ein Diagramm, daß, wenn durch maßgerechte Vorfertigung und sorgfältige Montage die möglichen ausführungsbedingten Exzentrizitäten klein gehalten werden können, die Tragfähigkeit solcher Wandplatten beachtlich erhöht werden kann. Nach dem von V a r s a n o gezeigten Beispiel ist es möglich, bei einer 14 cm starken, 2,6 m hohen Wand die Tragfähigkeit um ca. 19 % zu erhöhen, wenn die Exzentrizität nur um 1/2 cm gesichert verringert werden kann. Hier kann auch auf das Internationale Symposium "Bearing Walls" in Warschau im September 1975 hingewiesen werden, wo derartige Probleme diskutiert wurden.

Die von M e h l h o r n und S c h w i n g mitgeteilten Ergebnisse über die Tragfähigkeit von glatten und verzahnten Fugen sind sehr wertvoll, weil sie unter Auswertung von Versuchsergebnissen über eine theoretische Analyse zu einem allgemeingültigen Bemessungsansatz für den praktisch tätigen Ingenieur geführt haben. Für Trägheitsmomente von Wandplatten werden ebenfalls in praktisch verwertbarer Form Abminderungsfaktoren bei Berücksichtigung der Schubverformung der Vertikalfugen angegeben. Die zum Tragverhalten von Deckenscheiben aus Fertigteilen angedeuteten Ergebnisse von numerischen Untersuchungen sind leider im Beitrag noch nicht in der gleichen praktisch nutzbaren Form ausgewiesen. Man wird deshalb den hierzu angekündigten Veröffentlichungen mit einem Bemessungsvorschlag mit Interesse entgegensehen.

Zu den Aussagen von M e h l h o r n und S c h w i n g, daß die Steifigkeit der Deckenscheiben aus Fertigteilen geringer als die von Ortbetonscheiben ist, muß hinzugefügt werden,

daß aber für die praktisch vorkommenden Fälle bei konstruktiv zweckentsprechender Gestaltung die Deckenscheiben aus Fertigteilen den an sie gestellten Forderungen hinsichtlich einer ausreichenden Standsicherheit der mehrgeschossigen Gebäude völlig genügen.

Interessante Versuche mit verschiedenen mehrfach wiederholten Belastungen an außerordentlich breiten Fugen hat K a v y r c h i n e mit seinen Mitarbeitern durchgeführt. Für 18 cm breite, verzahnte Fugen mit Schlaufenverbindungen, die als vertikale Fugen in Wandscheiben Anwendung finden, konnten gegenüber bisher bekannter Grenzbeanspruchungen auch infolge wiederholter Belastungen keine negativen Auswirkungen erkannt werden. 6 Versuche mit glatten und 50 cm außergewöhnlich breiten Fugen, in welche von beiden Seiten gerade Bewehrungseisen einbinden, wie dies im Verkehrs- und Brückenbau vorkommen kann, zeigten unter wiederholter Belastung bei Biegung ein Verhalten wie ein monolithischer Stahlbetonträger und bei Längsschub mit Biegung ergab sich eine Grenzschubspannung von 1,13 N/mm^2. Man hätte sich gewünscht, daß die Versuchsergebnisse noch etwas umfassender, besonders im Vergleich zu analythischen Verfahren, im Beitrag beschrieben wären.

Bedauerlich ist, daß zu den anderen Problemen der Sicherheit und Stabilität von Elementen und vorgefertigten Bauwerken, die der Einführungsbericht herausstellt, keine Beiträge eingereicht wurden. Aber gerade solche Fragen

- wie können die erhöhten Qualitätssicherungen bei der Herstellung der Betonfertigteile in die Sicherheitskonzeptionen eingebaut werden ? oder
- wie können die rechnerischen Sicherheiten im Montagezustand für einzelne Elemente, Abschnitte und das gesamte Bauwerk gegenüber dem späteren Gebrauchszustand verringert werden ? oder
- mit welchen sicherheitstheoretischen Methoden werden Abweichungen vom zugrundegelegten idealisierten statischen System im Fertigteilskelettbau erfaßt ?

müssen ausreichend beantwortet werden, weil davon die Wirtschaftlichkeit der Fertigteilkonstruktionen entscheidend mit beeinflußt wird. Wir müssen doch besonders unter dem aktuellen Aspekt eines geringsten und zweckentsprechendsten Materialeinsatzes bei einem weltweit steigendem Baubedarf nicht so sicher wie möglich, sondern so sicher wie notwendig bauen. Es ist zu hoffen, daß die Forschung sich diesem Problem verstärkt zuwendet.

Comments by the Author of the Introductory Report

Remarques de l'auteur du rapport introductif

Bemerkungen des Verfassers des Einführungsberichtes

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Developments in Manufacture and Assembly

I will give a preliminary talk on Sub-theme VIb of Theme VI, Precast Structures. This sub-theme has the objective of discussing methods of manufacturing elements used in various precast structures and methods of connecting elements together to form assemblies.

The first step towards success with precast structures is the economical manufacture of elements of the specified shapes and dimensions, and which possess the required strengths, durabilities and degrees of watertightness. Numerous studies have accordingly been carried out regarding manufacturing techniques for each type of element so that the present situation in the world is that manufacturing methods are showing progress year by year. As a result, there are many elements for which top-class revolutionary techniques of current concrete technology are being utilized.

Designing and practices concerning such matters as joints between precast concrete elements, connections between precast concrete members, and connections between precast and cast-in-place concrete members are of the greatest importance in construction of assemblies, and much research has been done along these lines in the past.

There are six papers contained in the Preliminary Report which come under Theme VIb.

The paper of Messrs. Dratva and Gebauer discusses the results of studies to obtain high-strength concrete using portland cement clinker as concrete aggregate. The compressive strengths of concrete using ordinary aggregate and concrete using portland cement clinker are illustrated in the paper, and in the case of clinker the strength is approximately 80 N/mm^2 at the age of 28 days, which is 1.45 times the strength in case of ordinary aggregate. The strength in case of using clinker on crushing is even higher, being approximately 1.6 times that for ordinary aggregate. The effect of clinker on concrete is prominent also for flexural strengths as for compressive strengths. This is due to the great improvement in bonding properties between clinker and cement paste

brought about by hydration of the clinker.

It is concluded that the use of portland cement clinker is a way of readily obtaining concrete of high strength. Needless to say, however, examination of the economics will be necessary.

Mr. Dartsch's paper discusses means of increasing concrete strength at very young age. If it were possible to strengthen concrete of young age, stripping time could be shortened to facilitate reuse of forms, which would be advantageous since manufacturing costs of precast members could be reduced.

In the paper, comparisons of compressive strengths when short fibers of steel, glass and synthetic resin of lengths of about 3 cm are mixed in concrete at a ratio of just 0.6% by volume with compressive strength without fibers are illustrated. The effects of these short fibers on concrete strength at early ages within several hours of mixing are clearly indicated with strengths being as high as double the ordinary strength.

Furthermore, the paper states that the addition of fibers does not adversely affect the long-term properties of concrete, that practical addition quantities are 1% by volume or less, and concludes that this measure is economical.

Messrs. Murata, Okuyama and Kokubu discuss a method of increasing torsional strength of prestressed concrete piles.

This method conceives of providing a considerable quantity of spiral reinforcement in a prestressed concrete pile, and placing concrete containing a suitable amount of expansive component for chemical prestress in the peripheral direction. The outside mold is removed after steam curing in order to restrain expansion of concrete by the spirals and impart chemical prestress.

There is a graph in this paper in which angle of twist is indicated on the abscissa and torsional moment on the ordinate. The torsional cracking moment of a prestressed concrete pile with the combined utilization of chemical prestress is increased to 1.5 times that of an ordinary prestressed concrete pile. It is shown that ordinary prestressed concrete piles rapidly lose resisting capacities after development of cracks, but piles with both mechanical and chemical prestress resist torsion until yielding of spirals.

Messrs. Watanabe and Idemitsu discuss the use of a pretensioned cable truss they had devised for erection of a concrete arch consisting of precast blocks. A pretensioned cable truss may be considered as a suspension bridge with stiffening girders replaced by lower cables. The required pretensioning is provided by operating jacks inserted at the ends of upper cables and chain blocks installed at hangers. Cross beams for supporting concrete blocks are fixed to the hangers.

The arch is first divided into a number of parts in terms of width. For example, the arch is divided into three strips and the middle strip is first erected. The strips of the arch on either

side are erected after concrete of the middle strip has hardened. The concept in this case is that the total load would be carried by the middle strip of the arch with no load applied to the pretensioned cable truss. Connections are made by placement of cast-in-place concrete at joints between blocks, at joints between strips, and at the arch crown.

The authors surmise from results of model tests that work could be carried out much more rapidly and economically by utilizing pretensioned cable trusses for bridge erection than by using arch centers.

The paper by Messrs. Brachet, Olivier-Martin and Denis first discusses the results of investigation on prestress in pretensioned concrete girder bridges. Since steam curing is performed on girders of this type, losses in prestress occur due to expansion and relaxation of prestressing tendons, and these losses have been measured.

Losses in prestress were calculated from measured values of tensile forces acting on prestressing tendons obtained from readings of temperature-compensating strain gages, couplers with these gages attached to them being connected to the prestressing tendons and embedded in concrete. Along with other matters, it is stated that on comparing values measured at couplers set outside concrete with those at couplers embedded in concrete, the latter are found to show complex variations when steam curing is performed due to the influence of bond between concrete and prestressing tendons.

Loading tests of a continuous girder made by joining two precast simple girders through placement of concrete between them are next described, and it is reported that reactions due to loading agreed well with theoretical values. I believe the results of long-term loading tests will be described in today's discussion.

Mr. Machida describes performances in construction of two T-beam bridges and one truss bridge utilizing precast members of extremely high-strength concrete for the Sanyo super-express line of the Japanese National Railways.

This extremely high-strength concrete used good-quality aggregate and a water-cement ratio of approximately 0.30, greatly lowering unit water content by addition of a large amount of good-quality water-reducing admixture. In tests conducted prior to construction, it was confirmed that concrete of 28-day compressive strength of 750 kgf/cm^2 and slump of 12 cm could be obtained with a unit cement content of 485 kg/m^3, sand percent of total aggregate of 40% and the use of a polyaromatic sulfonate water-reducing admixture at a rate of 0.75% by weight of cement.

The truss bridge was a Warren truss of 45-m span as shown in Fig. 1. Fig. 2 shows connection of floor beams and Fig. 3 assembly of upper chords.

Based on these job performances, the rate of reduction in dead weight by the use of extremely high-strength concrete is indicated, while it is concluded that a truss bridge is economical since the construction period at the project site can be shortened.

SB

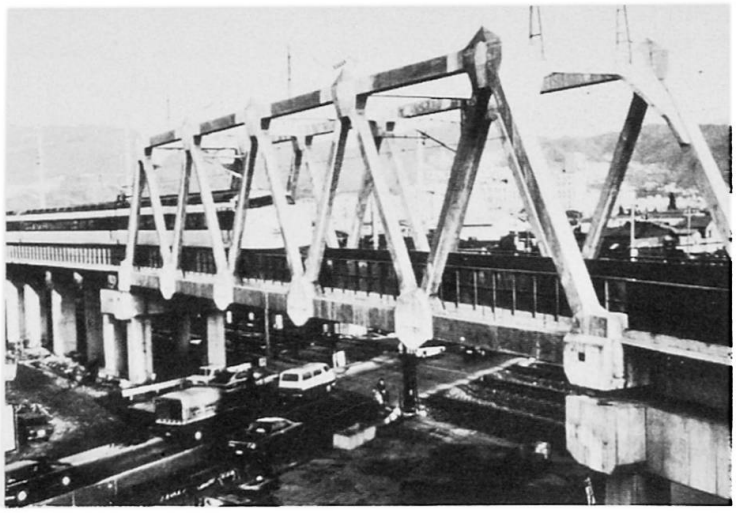

Fig. 1 — Concrete truss bridge of Sanyo super-exress line.

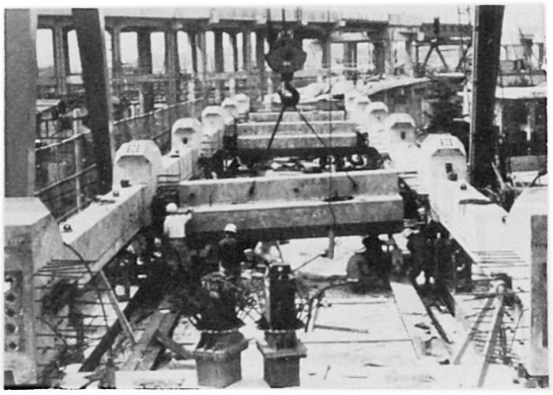

Fig. 2 — Connection of floor beams for truss bridge.

Fig. 3 — Assembly of upper chords for truss bridge.

Each of the six papers which I have just described provides information useful for solving the respective problems involved. However, the methods of designing and constructing a precast structure will differ greatly depending on the purpose for which the structure is used, the size of the structure, the topography, the geology, and other conditions of the jobsite. Accordingly, the problems discussed in these papers comprise only an extremely small part of the whole and there is naturally a considerable number of important problems remaining to be taken up.

The general advantages of the precast construction method are that it is possible to save manpower, that it is possible for rapid construction to be carried out, and that it is economical. However, it has come to be recognized that there are further advantages such as that obstruction of traffic and water flows can be reduced through elimination of shoring, and that disruption of

the environment accompanying construction can be lessened. Recently, it is often the case that these newly recognized advantages are of importance. Research works are being carried out in many countries throughout the world, structure by structure, in order that these advantages will be demonstrated. It is sincerely hoped that valuable discussions concerning the fruits of these works will take place amongst those gathered here today.

A prominent feature of precast construction in recent years is that precast elements have become large-sized with development of larger construction equipment. In order to deal with the larger sizes of elements, improvement in concrete quality, reduction of concrete weight, increasing the reliabilities of elements including accuracies of dimensions, and development of surer and more economical methods of joining elements are problems of particular importance which have arisen. I am especially looking forward to lively discussions ensuing in regard to these problems.

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Manufacture and Behaviour of Hollow Composite Members

Fabrication et comportement d'éléments composites creux

Herstellung und Eigenschaften von hohlen Verbundbauelementen

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1. Introduction

The use of hollow composite members consisting of relatively thin circular or rectangular steel tubes lined with concrete compacted by centrifugal force is conceivable as a means of economically improving the strength and ductility of precast concrete members. This will not only make manufacturing easier but should also handling easier due to a lightening in weight comparing to concrete filled steel tubes.

Hollow composite members are now used mainly as foundation piles, and wide utilization in bridge piers and columns and beams of buildings is looked forward to.

This paper discusses methods of manufacturing hollow composite members with high-strength concrete along with the structural behaviour of such members when subjected to bending.

2. Specimens and Test Procedure

2.1 Specimens

There are three structural types of hollow composite members, which may be cylindrical or rectangular in section, namely, 1) steel tube lined with concrete (Type I), 2) steel tube with reinforcement placed in the axial direction and lined with concrete (Type II), 3) steel tube with prestressing reinforcement lined with concrete and with prestress transferred after hardening of concrete (Type III). Comparison were made of the three structural types with specimens of hollow cylindrical section and in addition concrete filled steel tubes and hollow prestressed concrete members were tested as illustrated in Fig. 1.

The members were 4 m long and about 30 cm in diameter, and measured dimensions are shown in Table 1. The principal test

variables in the investigation were as follows; 1) wall thickness of hollow composite emmbers; 2) thickness of steel tube; 3) compressive strength of concrete; 4)bond properties between concrete and steel tube.

Axially welded mild steel tube were used in the investigation. The tensile and yield strengths of steel tube are reported Table 1. Round steel bar whose diameter 13 mm was used and its tensile and yield strengths were 4,340 kg/cm^2 and 3,150 kg/cm^2 respectevly. Prestressing wire whose diameter 7 mm was used and its tensile and 0.2 % proof strengths were 16,500 kg/cm^2 and 14,800 kg/cm^2 respecitively.

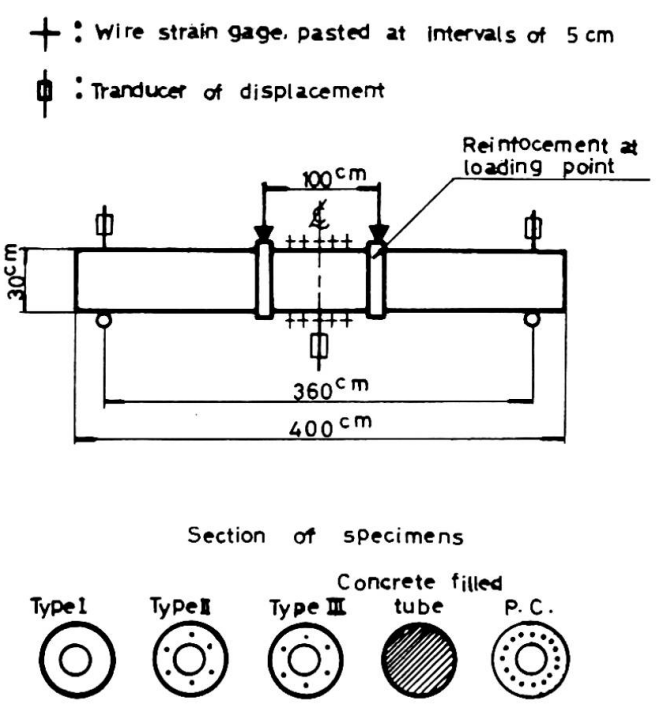

Fig.1 Specimen and gage arrangement

Normal portland cement and expansive component were used. A water-reducing agent whose main component was sodium salt of β-naphthalene sulfonic acid-formaldehyde condensate was used. Crushed stone whose maximum size was 20 mm and river sand were used as aggregates.

The base mix of concrete was as follows: water-cement ratio

Table 1 Specimens and test results

Specimen	Type of Specimen	Type of concrete	Dimension		Steel tube			Concrete	Ultimate bending strength		
			Outer diameter mm	Wall thickness mm	Wall thickness mm	Yield strength kg/cm^2	Tensile strength kg/cm^2	Comp. strength kg/cm^2	Observed tm	Calculated tm	Observed / Calculated
A	I	Expansive	298	28	3.15	3,220	4,620	844	12.08	13.21	0.91
B	I	Expansive	297	41	3.17	3,220	4,620	844	13.03	13.96	0.93
C	I	Expansive	296	58	3.19	3,220	4,620	1,024	13.88	14.21	0.98
D	Filled tube	Expansive	297	—	3.10	3,220	4,620	823	13.07	13.52	0.97
E	I	Expansive	298	60	3.13	3,220	4,620	458	13.26	12.39	1.07
F	Filled tube	Normal	297	—	3.18	3,220	4,620	426	11.44	11.45	1.00
G[2)]	I	Expansive	297	68	3.12	3,220	4,620	1,072	8.78	14.28	0.60
H-1	I	Normal	297	64	3.19	3,220	4,620	1,072	14.17	14.38	0.99
H-2	I	Normal	297	69	3.18	3,220	4,620	1,050	13.65	14.37	0.95
I[3)]	I	Expansive	299	61	2.92	2,160	3,640	980	10.95	10.33	1.06
J[3)]	I	Expansive	298	61	2.84	2,160	3,640	981	10.60	10.40	1.01
K	I	Expansive	297	59	1.36	2,830	3,730	940	6.23	6.46	0.96
L	I	Expansive	297	61	2.26	2,030	3,550	940	7.61	7.79	0.98
M	I	Expansive	297	61	4.46	3,040	3,870	947	15.21	16.51	0.92
N	I	Expansive	299	71	6.35	2,850	4,840	947	28.34	26.50	1.07
O	II	Expansive	297	64	3.18	3,220	4,630	929	16.97	18.50	0.92
P	II	Expansive	298	61	3.19	3,220	4,620	924	16.77	17.25	0.97
Q	III	Expansive	298	61	3.20	3,220	4,620	1,034	17.03	17.49	0.97
R	P.C.	Normal	302	58	—	—	—	1,002	10.92	10.30	1.06
S	P.C.	Normal	301	62	—	—	—	1,002	4.68	4.65	1.01
T-1	I	Normal	296	61	4.05	2,960	3,730	1,071	16.40	16.02	1.02
T-2	I	Normal	296	63	4.04	3,040	2,820	1,018	16.30	16.10	1.01
U	I	Expansive	296	70	4.06	2,950	3,740	1,128	16.70	16.85	0.99

Note: 1) All specimens were autoclaved but F. 2) Paraffin was coated about 2mm inside of steel tube of G. 3) Chequered plate was used for I and J. Rugged face was out side for I's tube and was inside for J's tube.

0.32, sand-aggregate ratio 42 %, cement content 404 kg/cm², content of expansive components 26 kg/m³, water-reducing admixture 1.6 % wt. of cement, slump 12 cm. The water-cement ratio fell to approximately 0.27 by the time the centrifugal compaction had been completed. The compressive strength was about 900 kg/cm² after autoclaving. Concrete strengths at the time of testing are reported for each member in Table 1 as an average of three 20 x 30 cm centrifugally compacted hollow cylinders which was autoclaved in the form.

Specimens were manufactured following process:

1) Steel tube or steel tube with reinforcement or prestressing reinforcement placed in the axial direction was set in the form.
2) Concrete was placed in the steel tube using concrete pump and centrifugal compaction is performed for 7 minutes. The maximum speed was 1,090 r.p.m..
3) Curing by atmospheric pressure steam at a maximum temperature 50 °C was carried out for about 12 hours.
4) Members were stripped at the next day. Prestress was introduced into concrete and steel tube in the case of type III.
5) Stripped member was cured by high-pressure steam for 9 hours at maximum pressure at 10 atm. (180 °C).

2.2 Test Procedure

All specimens were supported on rollers at 3.6 m centers and were loaded at two points 0.5 m either side of the midspan, as shown in Fig.1. The ultimate load and load-deflection chracteristics were observed. In addition longitudinal and transverse strain of steel tube were recorded thoughout the test. The arrangement of gages is shown in Fig.1. The wire strain gages and strain meter which were able to measure great strain to 10 % were used in the test. Outer face of specimens were reinforced for 10 cm long at loading point, in order to prevent from the falling of concrete by shearing force.

3. Test Results

Typical load-deflection diagrams of hollow composite members are illustrated in Fig.2. It was verified that the properly made hollow composite member irrespective of type has greater ductility than the prestressed concrete member and that

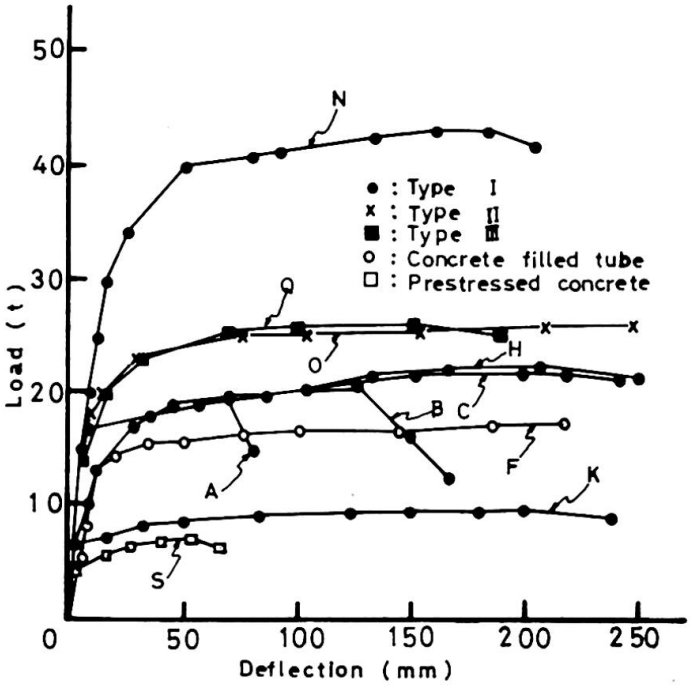

Fig. 2 Load – deflection diagram

structural behavour of hollow composite member was equal to that of concrete filled steel tube. The inside view of tested member is shown in Fig.3. It is shown in this figure that the concrete at compressive zone was completely failed and that the concrete at tension zone developed many cracks and that concrete was not thrusted out to the inside of the member. In case of the member whose wall thickness was thin, concrete at compressive zone was thrusted out into the inside, so that it had not so much ductility.

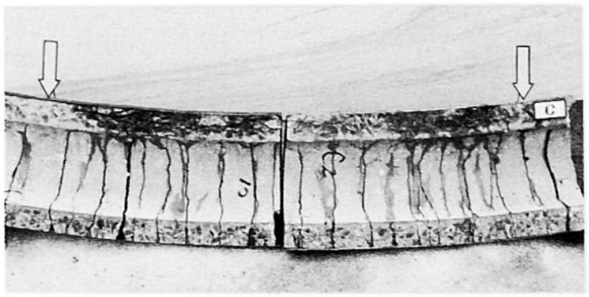

Fig. 3 Inside view of tested specimen

It is conceivable that complete interaction takes place between the steel and the concrete in the case of hollow composite member lined with normal concrete. This is thought to be because even if there is no bond between steel tube and concrete, when the member is deformed due to external force, the concrete is restrained by the steel tube and frictional force is produced between the two, which acts as an apparent bond.

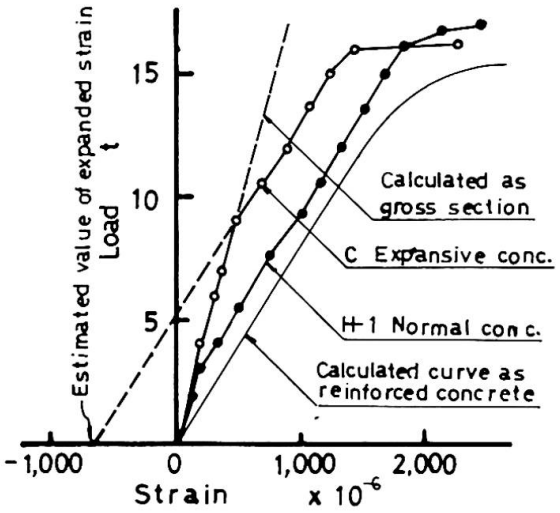

Fig. 4 Load-strain diagram

By using expansive concrete the deflection in the elastic range will be improved. This is because chemical prestress is induced in the member as shown in Fig.4. The chemical prestress induced in the specimen C is estimated about 92 kg/cm^2.

Fig.5 shows the relation between longitudinal strain and ratio of transverse strain and longitudinal strain. The ratio at compression side shows neary fixed value till the longitudinal strain is 1,000 x 10^{-6}, but it increases remarkably when the longitudinal strain exceeds 1,000 x 10^{-6}. This is thought to be because the concrete at the compression zone is failed and steel is expanded by the concrete.

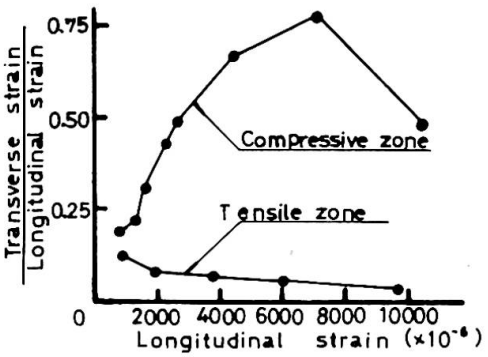

Fig.5 Relation between longitudinal strain and $\frac{\text{transverse strain}}{\text{longitudinal strain}}$

At the tension side, the ratio decreases as the longitudinal strain increases. The value is smaller than the poisson's ratio of steel. This shows that the steel tube is expanded by concrete also at the tension side.

The typical computed load-strain curves are shown in Fig.7 assuming as follows:

1) The concrete has no tensile strength;

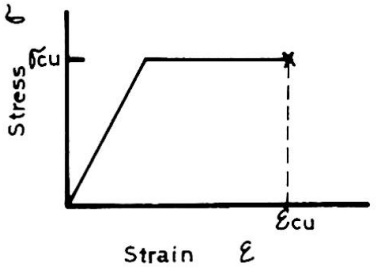

Fig.6 Assumed stress-strain relationship for concrete

2) The stress-strain relationship of concrete under compression is assumed as Fig.6;
3) Uniaxial state of stress for the steel is assumed. And the observed stress-strain relationship is used;
4) Plain section remain plain after bending.

Fig.7 shows that the observed compressive fiber strain was more than 10,000 x 10^{-6} and that the calculated and observed compressive and tensile fiber strain were roughly equal. Fig.8 shows the relation between ultimate bending strength calculated using the four assumptions and observed ultimate bending strength. The ultimate compressive fiber strain was used 10,000 x 10^{-6} for the hollow composite members and 3,000 x 10^{-6} for the prestressed concrete members.

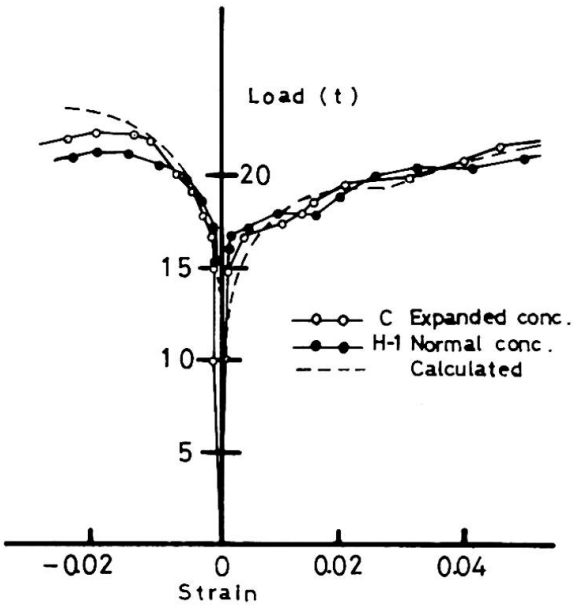

Fig. 7 Load – strain curve

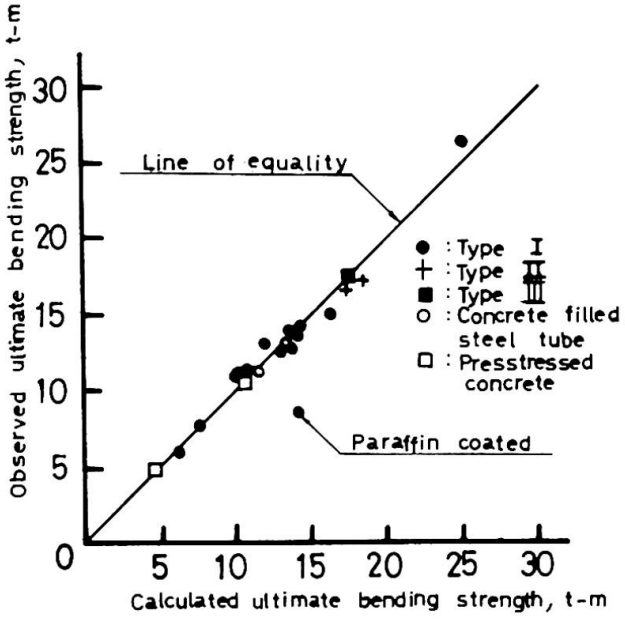

Fig 8 Relation between observed and calculated value of ultimate bending strength

4) Conclusions

The structural behaviour of this type of member indicated by the experiments may be summarized as follows:

1) It was verified that the hollow composite member has a high bending strength and great ductility, and this is thought to be due to the prevention of buckling of the steel tube, to the continuance of the united action of the concrete with the steel even when the internal concrete in the compression zone had become greatly deformed by the restraint of the steel tube and arch action, and to the fact that concrete in the

tension zone did not spall even after crack development.

2) Complete interaction takes place between the concrete and the steel tube of hollow composite member, lined with normal concrete without treating the inside of steel tube. The use of expansive concrete in the hollow composite members introduces chemical prestress to the concrete and improves the structural behaviour of the members at the elastic range.

3) The ultimate bending strength of this type of member could be calculated ignoring the tensile strength of concrete, assuming that the section would at all times maintain a plane, and that the maximum compressive fiber strain is 10,000 x 10^{-6}.

SUMMARY

It was verified that hollow composite members consisting of relatively thin circular steel tubes lined with concrete compacted by centrifugal force has a high bending strength and great ductility. The ultimate bending strength of this type of member could be calculated ignoring the tensile strength of concrete, assuming that the section would at all times maintain plane, and that the maximum compressive fiber strain is 10^{-5}.

RESUME

Il a été vérifié que les éléments composites creux formés d'un tube circulaire en acier mince revêtu de béton compacté par centrifugation, ont une grande résistance à la flexion, et une haute ductilité. L'effort à la flexion maximum pour ce genre d'éléments, peut être calculé sans tenir compte de la résistance à la rupture du béton, en supposant que la section reste tout le temps plane et que l'effort maximum de compression dans la fibre est égal à 10^{-5}.

ZUSAMMENFASSUNG

Die hier behandelten hohlen Verbundbauelemente bestehen aus einem Stahlrohr verhältnismässig kleiner Wanddicke und Beton, welches auf die Innenseite des Rohrs durch Schleudern aufgebracht wird. Es wurde nachgewiesen, dass diese Bauelemente eine grosse Biegefestigkeit und Duktilität besitzen. Ihr Bruchmoment kann unter Vernachlässigung der Betonzugfestigkeit unter der Voraussetzung ebenbleibender Querschnitte und einer maximalen Druckstauchung von 10^{-5} rechnerisch ermittelt werden.

Wirtschaftliche Realisierung grosser Deckenspannweiten bei hochinstallierten Gebäuden

The Economic Use of Large Spans in Buildings with Service Floors

Réalisation économique de grandes portées pour des immeubles hauts

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Die sich überschlagende Entwicklung technischer Einrichtungen (z.B. Medizin, Fernsehen etc.) erfordert Bauten, deren Einrichtung und Installation leicht auf- und abbaubar sind. Wünschenswert sind große Freiflächen und begehbare Installationsgeschosse (Bild 1).

Bild 1. Neubauten des Klinikums Aachen im Ausbauzustand

Es wurden zwei Systeme untersucht, bei dem einen wird die Feldzone frei überspannt, bei dem anderen mit Hilfe eines Zwischenjochs (Bild 2). Die zweite Lösung wurde nur zum Teil mit Installationsgeschossen ausgeführt.

Für die freie Überbrückung der großen Spannweiten wurde eine Höhe für die Deckenträger und Installationsgeschosse von 2,40 m = 1/14 der Spannweite gewählt. In Längsrichtung werden die Träger

Für die Klinik in Aachen mit 1600 Betten erwuchs aus den Funktionsbedingungen für das Hauptgebäude ein Baukörper von 225 m Länge, 135 m Breite und 54 m Höhe - ca. 1,3 Mio m^3 umbauter Raum. Er gliedert sich in vier Kernzonen von je 7,20 m Breite und die dazwischen liegenden drei 33,60 m breiten Feldzonen. Die in Längsabständen von 43 m stehenden 24 massiven Kerne (6 je Kernzone) steifen das Tragwerk aus.

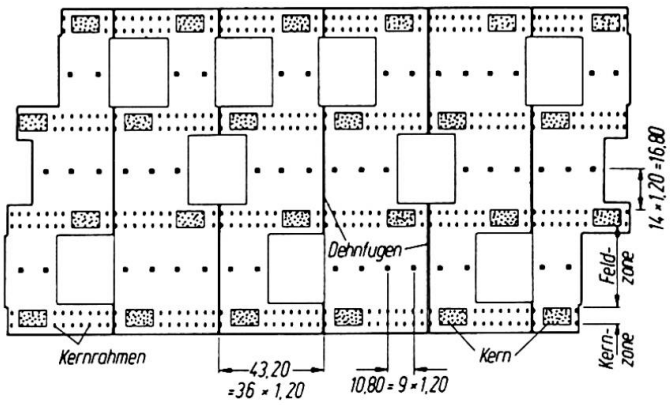


Bild 2. Grundriß des Hauptgebäudes

zur Führung der Trassen durchbrochen. Die ovalen Öffnungen in Trägermitte dienen dem Durchstieg (Bild 3).

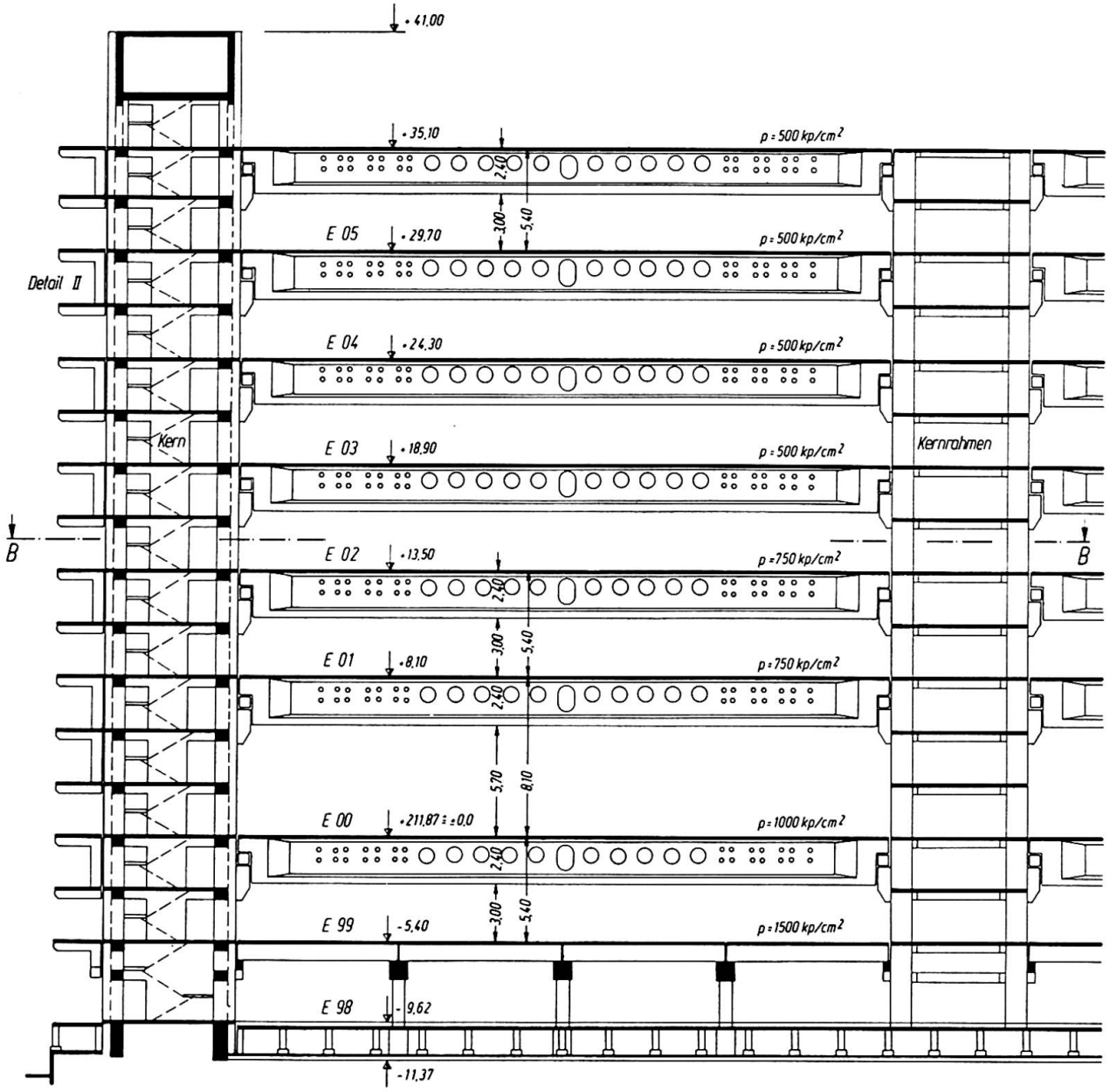


Bild 3. Querschnitt durch eine Außenfeldzone mit angrenzenden Kernzonen (Kerne) links bzw. Kernrahmen rechts. Tragsystem "Große Spannweite"

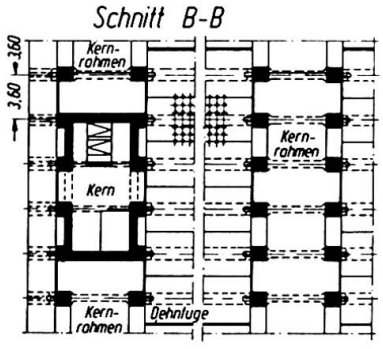


Bild 4. Grundriß der Ebene E 01 im Bereich einer Außenfeldzone mit angrenzenden Kernzonen

Zwei U-förmige Teile bilden je einen Kern (Bild 4). In zehn Tagen wurden je zwei von ihnen in Gleitschalung erstellt (Bild 5). Sie und die zwischen ihnen in Abständen von 3,60 m angeordneten Kernrahmen nehmen über Konsolen die Deckenlasten auf. Die außen liegenden Kernzonen werden um 2,40 m breite Randstreifen erweitert, deren Decken und Kragarme vorgefertigt sind, um sie bei einer späteren Erweiterung

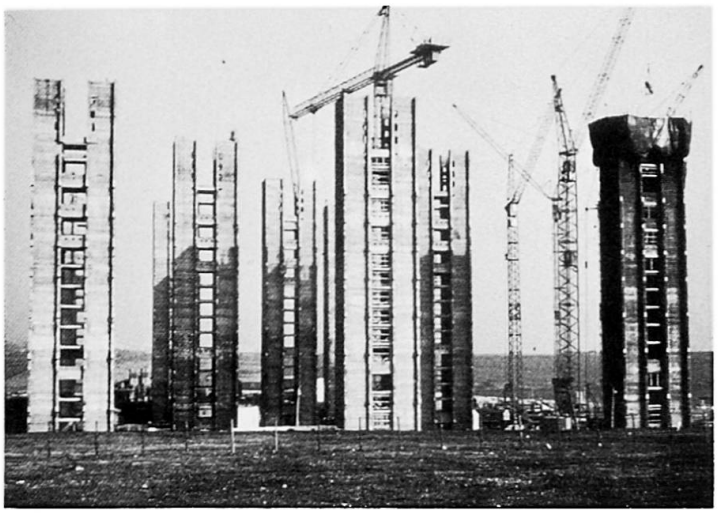


Bild 5. Fertiggestellte Treppenhauskerne

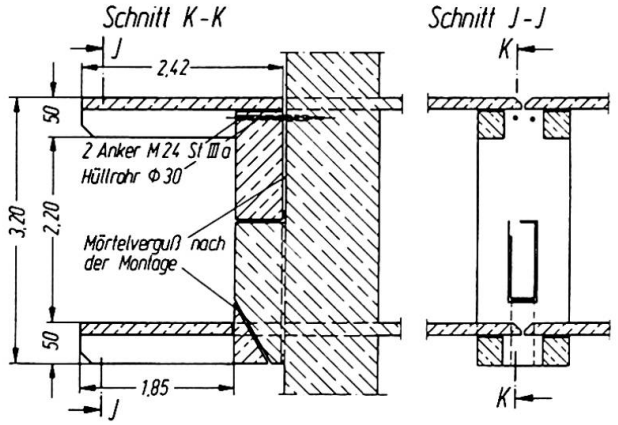
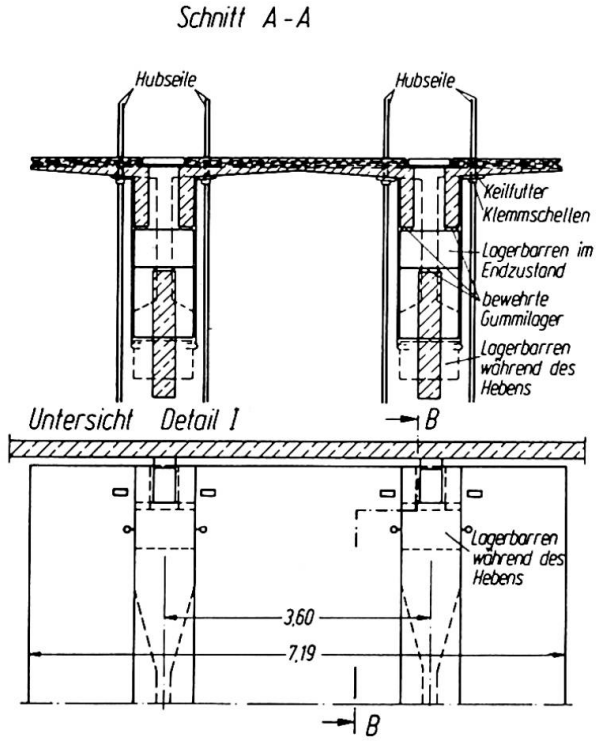


Bild 6. Ausbildung der Randstreifen im Bereich der außenliegenden Kernzonen (vgl.Bild 3) mit vorgefertigten Kragarmen und Deckenplatten



leicht demontieren zu können (Bild 6). Sie sitzen auf für diesen Fall bereits vorgesehenen Trägerkonsolen und sind mit Gewindestäben an Kernwänden und Stützen angeschraubt, um sie leicht entfernen und wieder verwenden zu können.

Die Decken bestehen aus beschränkt im Spannbett vorgespannten 33,50 m langen und 2,40 m hohen $\top$ -Trägern aus Beton B450, bemessen für eine Nutzlast von 1000 kp/m^2 (Bild 7). Die Deckenplatte,

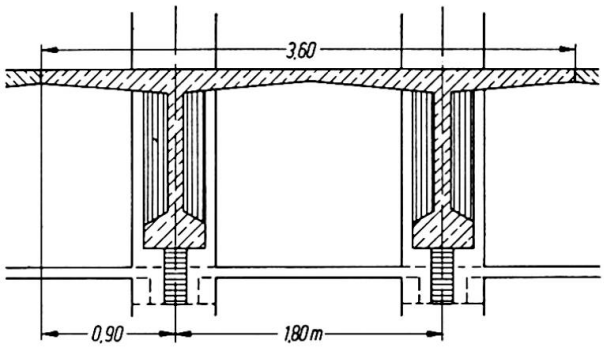


Bild 7. Querschnitt der 33,50 m langen Deckenträger in den Feldzonen

im Mittel 20 cm dick, kragt während der Montage beidseitig aus und wirkt nach Fugenverguß durchlaufend und als horizontale Scheibe. In dem 80 cm breiten Trägerfuß sind bis zu 125 ovale Spanndrähte untergebracht.

An den Trägerenden ist der sonst 20 cm dicke Steg auf Fußbreite verdickt und mit 5 Spannstäben Ø 26 vertikal vorgespannt. Die Auflagerkonsolen sind gabelartig ausgebildet.

Bild 8.
Ausbildung der Trägerenden im Hub- und Endzustand (Detail I, Bild 9)

Bauablauf und Montage wurden wie folgt vorgesehen:

1. Schritt Herstellung der Pfahlgründungen unter den Kernen und der Flachgründungen unter den Kernrahmen. Gleichzeitig Aufbau von zwei Feldfabriken an den Stirnseiten des Bauwerks.

2. Schritt Fertigung der Kerne in Gleitschalung und Montage der in stationären Werken hergestellten Kernrahmen.

3. Schritt Nach Aufbau der Feldfabriken Herstellung der Deckenträger im Spannbett mit Dampfhärtung.

4. Schritt Schienentransport der Deckenträger unter die Einbaustellen; Montage der hydraulischen Senkheber auf den Köpfen der Kerne bzw. Kernrahmen.

5. Schritt Hub der Deckenträger (Bild 9).

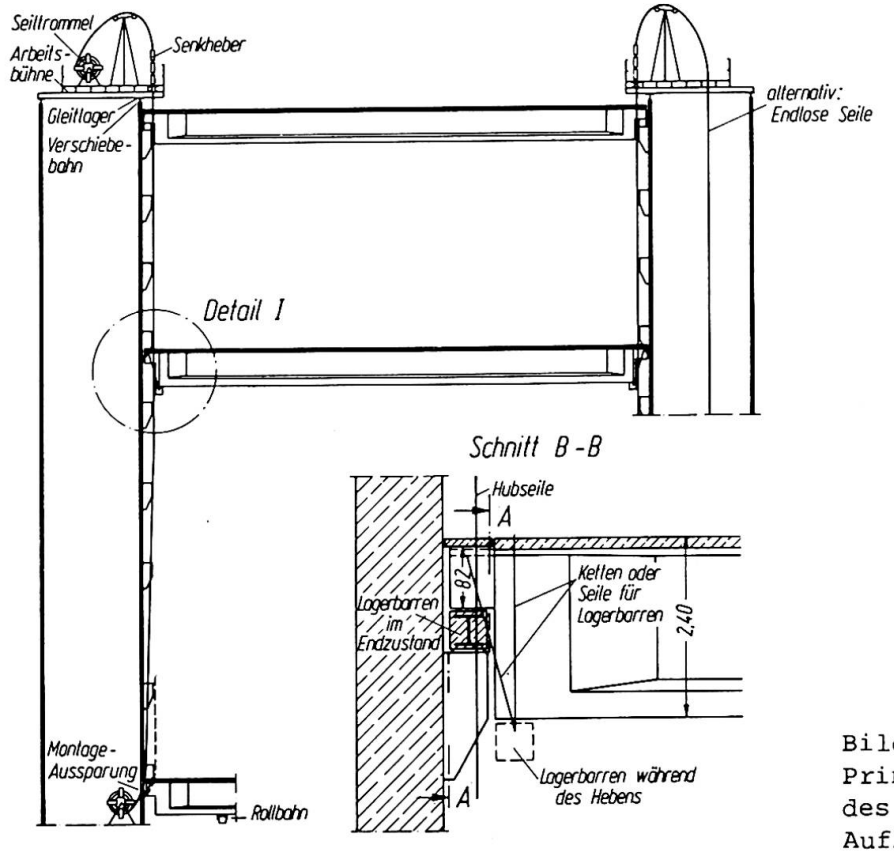


Bild 9.
Prinzipielle Darstellung des vorgesehenen Hub- und Auflagerungsvorganges

Beim Hub der Deckenträger gleiten die Gabelkonsolen an den Stützenkonsolen vorbei. Die vorgesehene Höhe wird etwas überfahren, damit die Lagerbarren versetzt werden können. Letztere hängen während des Hubvorganges an den Enden der Träger an je einer schräg- und einer vertikal gerichteten Kette. Durch Lösen der vertikalen Ketten schwenken die Barren zwischen die Konsolen und ermöglichen das Absetzen des Trägers auf bewehrte Gummilager.

Bei gutem Wetter hebt man alle übereinanderliegenden Träger nacheinander, bei schlechtem Wetter zuerst alle Dachträger, um unter ihrem Schutz alle weiteren Hub- und Montagearbeiten durchführen zu können. Hierzu müssen die Seile der Senkheber hochgezogen werden, damit die Hubausrüstung auf der fahrbaren Arbeitsbühne verschoben werden kann. Parallel zu den Hubvorgängen werden die

Ringanker verschweißt und die Fugen vergossen. Unter Berücksichtigung einer Hubgeschwindigkeit von 5 m pro Stunde, der Arbeitszeit für das Absetzen der Träger und das Verschieben der Arbeitsbühne erfordert das Heben von sieben übereinanderliegenden Trägern 85 bis 95 Stunden. Mit diesen Werten ist es möglich, Montagegeschwindigkeiten von 18 t Fertigteilen = 18 m^2 Deckenfläche je Stunde zu erzielen. Nach Einspielen der Mannschaften erscheint mit zwei Hubausrüstungen in zwei Feldzonen eine Montageleistung von 400 t je Tag möglich.

ZUSAMMENFASSUNG

Für die Neubauten der Medizinischen Fakultät der Universität Aachen wurden zwei Entwürfe erarbeitet. Der eine mit einer Deckenspannweite von 16,80 m wurde ausgeführt. Zur Erzielung von funktionellen und installationstechnischen Vorteilen - z.B. separate Installationsgeschosse - stützte sich der zweite Entwurf auf eine Spannweite der Deckenträger von 33,60 m. Geschosshohe π-Deckenträger überbrücken diese Spannweiten. Der beabsichtigte Bau- und Montageablauf mit Seil-Senkhebern von 250 Mp Hublast je Trägerseite wird beschrieben.

SUMMARY

Two designs, with different span lengths were studied for the new building complex of the Faculty of Medicine at the University of Aix-la-Chapelle. The alternative with the shorter span of 16.80 m was executed. The second design made use of storey-high double T-beams spanning 33,60 m, in order to reduce the number of prefabricated elements and to facilitate erection and accessibility for replacement of installations in service floors. The article describes the envisaged method of construction using 250-ton gear for lifting and lowering.

RESUME

Pour les nouvelles constructions de la Faculté de Médecine de l'Université d'Aix-en Chapelle deux projets ont été élaborés. L'un d'eux, celui d'une portée des dalles de 16,80 m fut exécuté. Pour obtenir des avantages fonctionnels et techniques au point de vue des installations - p.ex. des étages de service séparés - le deuxième projet se basait sur une portée des poutres du plafond de 33,60 m. Des poutres en double T de la hauteur d'un étage franchissent ces portées. Le procédé de construction et de montage prévu au moyen de dispositifs de levage et d'abaissement à câbles de 250 Mp de charge de levage est décrit.

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VI

A New System for Prefabricated Slab Elements

Un nouveau système d'éléments de dalles préfabriquées

Ein neuartiges System für vorfabrizierte Deckenplatten

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1. Introduction

On the basis of a demand issued by the owner of the planed "Shopping Center, Glatt AG" in Zurich, Switzerland, our office of consulting engineers investigated the possibility of erecting the projected buildings (tall office building, shopping and parking, a total of approx. 1'000'000 cb.m of buildings) with prefabricated concrete elements, and the building time which could thereby be gained. In conformity with the project - already granted-, the column to column span of 8.0/8.0 meters, as well as the maximum store height had to be observed. Under these conditions, normal precast slabs with conventional primary and secondary elements would have met with difficulties, in particular in connection with the Parking, due to the excessive building height. Thus, a new system without girders had to be developped.

2. Slab Elements

As solution we selected a system admissible for transportation by road consisting of slab elements, of which two each could be connected at the center of the span (their own size amounting to 8x4 m), by means of a steel coupling. The main characteristic of this system was the inclusion of the main beam into the secondary elements together with the coupling of the main beams at the center of the span during the phase of erection. (s.illustration no.1). Thus, two slabs were coupled into a span of 8x8 m between the columns (Fig.2, Isometrics of Element Construction). By an adequate positioning of the two couplings it was possible to obtain an uniform type of elements and to avoid a division into "right-side" and "left-side" elements. In addition, the position of the couplings at the center of the span offers the following advantages: reduction of the couplings to two per field, a minimum of shearing forces, an unequivocal moment direction at this particular spot, and one type of element only per field.

Fig 1: Bottom view of slab elements. At the center of the span, the joint where the elements are coupled is visible.

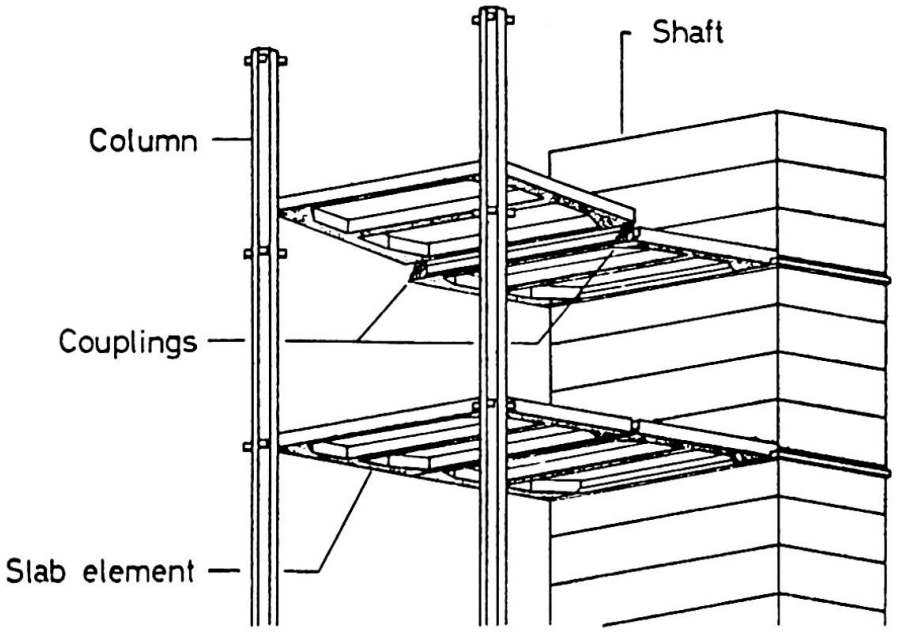

Fig. 2 : Isometrics of Element Construction

The coupling consists of different parallel interlocking steel laminae which are coupled by means of two counteracting cones in the lower (tension) and the upper (compression) area of the slab (see fig. 3 and 4).

The reinforcing rods are directly welded to the laminae, made of St52-3. Each coupling half is factory - made together with the welded reinforcement rods, and is placed into the form as a complete unit.

The slab elements with a slab thickness of 4 cm and a beam height of 35 cm are fitted after assembly with a concrete layer of 7cm thickness, containing a reinforcement with a static effect. The total construction height therefore amounts to 46 cm.

3. Tests

A main element of the tests was the coupling of the slab elements. On the basis of a list of priorities, partly resulting from experiences gathered from other already realized constructions, the following prerequisites took shape:

1. Safety of the connections
2. Adjustability of the horizontal and the vertical plains
3. Short assembly time
4. Immediate carrying capacity, i.e. no setting time for concrete or similar properties, no stressing operations with high quality material from the technical point of view and adequate operating personnel during the assembly process.

This novel and now patented system has been verified through detailed series of tests, among other also carried out by the Swiss Federal Laboratory for Testing Materials and Research (EMPA), as to carrying capacity and fatigue.

The following investigations have been carried out in particular:

1. Optical investigation pertaining to mechanical stress of the main coupling element with corresponding perfectioning of its shape;
2. Dynamic tests of the necessary welding connections under different loads;
3. Extended tests with test girders 1:1 with repeated-stress test (vibration test) and final rupture test.

Deflections were essentially investigated under the following loads:

1. Single dead weight momentum;
2. Double dead weight momentum;
3. Maximum momentum;

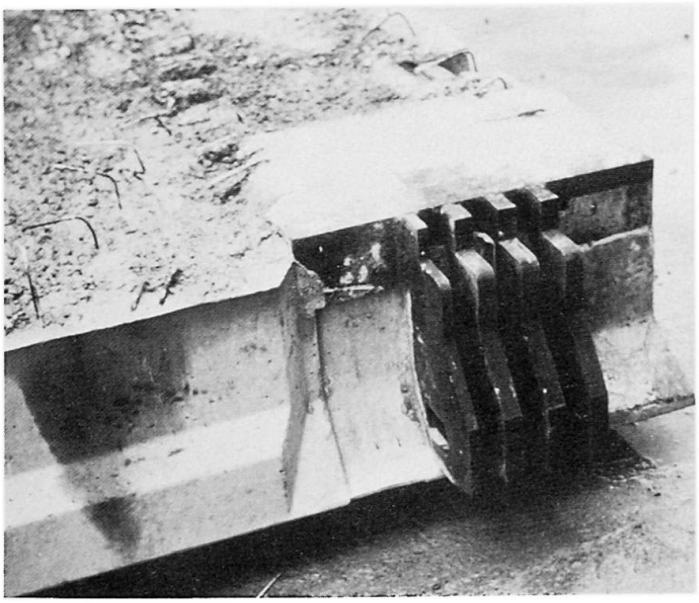

Fig.3: Coupling of Elements.

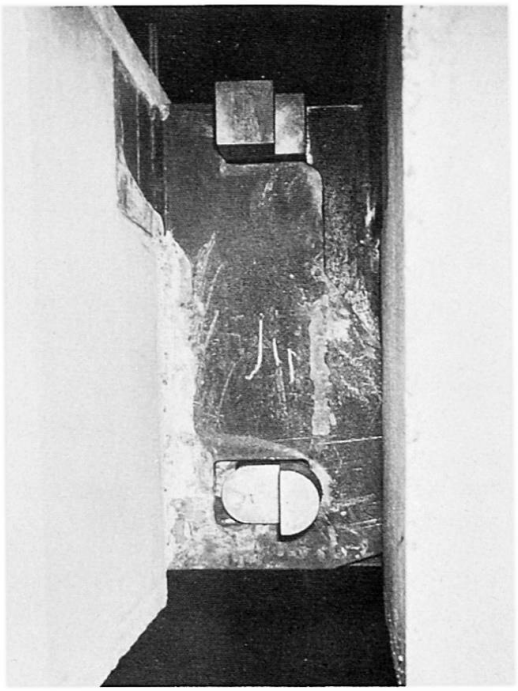

Fig.4: Element Coupling, lateral view showing also the cones on the compression-resp. the tension-side (picture taken during the tests at EMPA).

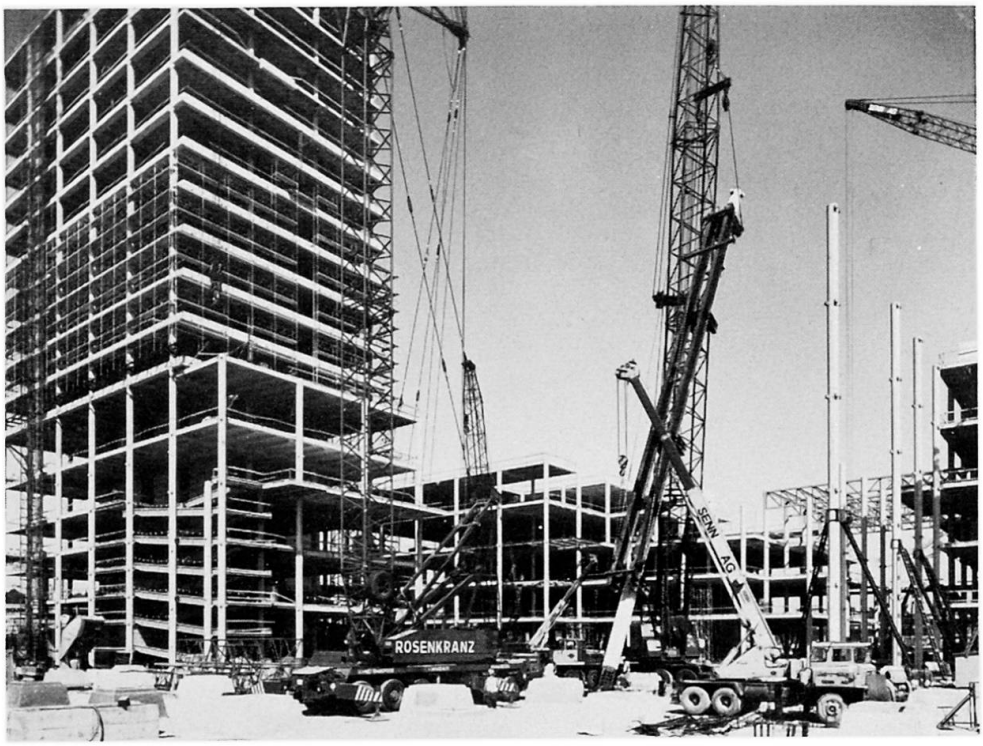

Fig.5: Erection of the Elements.

4. 200'000 oscillations over live load width;

5. Increase of load up to rupture.

As expected, during the first load we met with the cone pressure effect. The deflection at live load width, e.g., amounted during first load to 6 mm, whilst at second load it already decreased to 3 mm. A comparative calculation concerning the E-module of a normal reinforced concrete beam showed a value of E = 295'000 kg/cm2 for second and further loads. An increase of deflection after the 200'000 oscillations attained the value of less than 2 mm, i.e. approx. 1/3100. (For reasons of availability of space, the test beam had a span width of 6.2 m in the test plant).

The final rupture test warranted the following results: up to a load of M = 50 mpm deflections remained linear to a large extent. The stretching-strain limit of reinforced concrete steel was reached between 50 and 55 mpm. By a further consolidation of the coldworked Roll-S-steel, an increase of the momentum stress to 70 mpm became possible, whereafter rupture took place under a load press, i.e. not in the coupling (maximum momentum and shearing force crack). During the subsequent disassembly of the coupling, the cones showed compressed surfaces. Deflections or flow lines on the laminea could not be found.

4. Assembly of the Elements

The sequence of assembly of the elements was carried out in vertical direction. Before application of the covering concrete, the coupling area was reinforced, i.e. supported by especially developped, hydraulic braces, so that the live load and weight of the covering concrete act as a continuous system. By means of an immediate disengagement of the brace after assembly it becomes possible for every slab to be loaded with the dead weight of the ceiling above, in addition to the proper slab weight itself. Thus it became possible to compensate cone compression arising during the first load of the coupling cones over a wide range of the subsequent load width.
From the foundations upward, the entire construction was built with prefabricated elements (s. illustration no. 5). Besides the aforementioned slab elements, also the columns having a height of up to 28 m, and the shafts required for vertical connections (elevators, piping), as well as for the stability of the buildings have been prefabricated.

Assembly of these approx. 12'000 elements started in May 1973 and was terminated for all buildings within one and a half year. This made it possible to finish the construction 10 months earlier than foreseen by the orginal conventional project.

SUMMARY

For the prefabrication of a Shopping-center and high-rise office building in Zurich, a new system of prefabricated slabs had to be found. For the column to column-span of 8.0/8.0 m a slab-element of 8.0/4.0 m was developped; two slab-elements had to be connected in the center of the span with steel-couplings. With this kind of prefabrication the assembly time for the building could be reduced by 10 months.

RESUME

Pour l'exécution d'un centre d'achats et d'une tour commerciale près de Zurich, il était nécessaire de trouver un nouveau système d'éléments de dalles préfabriquées. Pour une distance entre les colonnes de 8.0/8.0 m, on a introduit deux éléments de 8.0/4.0 m qui sont assemblés au milieu de portée par des raccords d'acier.
Une telle construction a permis de réduire le temps de construction de 10 mois.

ZUSAMMENFASSUNG

Für die Vorfabrikation eines Shopping-Centers mit Bürohochhaus bei Zürich, musste ein neues Elementdeckensystem mit Stützenabstand 8.0/8.0 m errichtet werden, das aus 8/4 m grossen Deckenplatten besteht, von denen je zwei in Feldmitte zusammengekoppelt wurden.
Durch diese Vorfabrikation konnte gegenüber dem ursprünglichen vorgesehenen Projekt 10 Monate Bauzeit eingespart werden.

Application of Chemical Prestress to Precast Concrete Box Culverts

Application de la précontrainte chimique aux éléments en béton fabriqué de conduites souterraines

Anwendung chemischer Vorspannung bei vorgefertigten Kanälen mit kastenförmigem Querschnitt

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1. Introduction

In Japan, with the extremely great increase in sewer line works and construction of underground utilities ducts, the use of precast concrete box culverts has increased rapidly, and 600,000 tons of products were manufactured in 1975. Since permissible crack widths in box culverts are severely restricted to insure the durability, the application of chemical prestress was considered. Chemical prestress is introduced in a reinforced concrete member by expansive concrete overcoming the restraint by steel. The amount of chemical prestress differs greatly if steel quantities and arrangements vary. In general, as the quantity of restraining steel increases, the amount of chemical prestress increases as shown in Fig.2, although the elongation of restraining steel decreases.

Almost as soon as an expansive admixture began to be on the market, chemical prestress was applied in precast products of pipe form[1]. Regarding the products of box form, chemical prestress was not applied until the end of 1973. From the test results of small sized frames, conducted in 1972 and described in this paper, it was confirmed that the application of chemical prestress to box culverts would be promising. Then, chemical prestress was first applied to the actual products of box form in Tsurumi Concrete Company where about 100,000 tons culverts being manufactured annually, after carrying out the quality control tests of expansive concrete and the loading tests of actual products. In 1975, chemically prestressed concrete products reached 50 % of the total products in the company, and it is expected to reach 75 % in 1976.

Concerning the application of chemical prestress, a question

Fig.1 Precast concrete box culvert

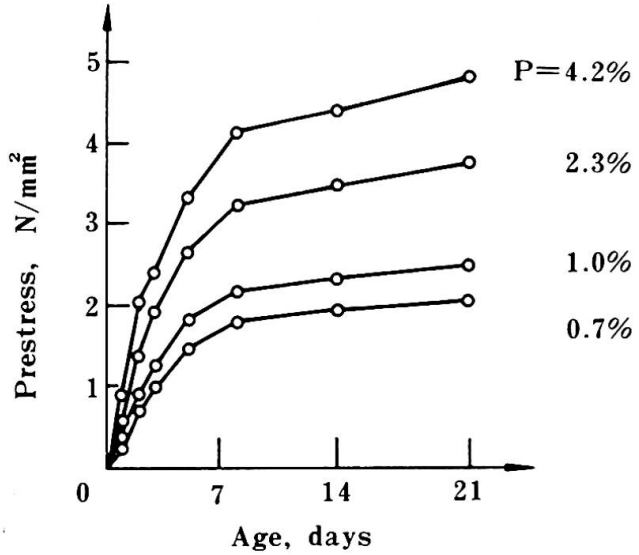

Fig.2 Increase in chemical prestress (p is percentage of steel in concrete)

was raised whether the effect of prestress might be decreased due to creep and shrinkage of concrete, eventually becoming almost completely lost, since the extent of prestress, which would be effectively utilized, would be about 3 N/mm^2 at most. The authors[2] have insisted that the effect of chemical prestress introduced in a member buried underground might last for a long period of time, because

(1) amount of chemical prestress in a member exposed to be weathered but not loaded did not change so much for a year,

(2) amount of chemical prestress in the tension side of a member, which is important for cracking resistance, did not decrease due to the external moment, and

(3) shrinkage of concrete might be small enough to be neglected for a member buried underground.

At this time, in order to confirm our insistence, experiments on actual products were carried out.

2. Behavior of Chemically Prestressed Concrete Box Culverts

2.1 Behavior of Beams

The increase in strain of a tensile reinforcing bar in a chemically prestressed concrete beam when external moment acts is extremely small compared with that in an ordinary reinforced concrete beam[3]. An example is shown in Fig.3. This reduction of strain is because of the chemical prestress, and the degree of the reduction is corresponding to the expansive strain produced in the bar during expansion of concrete. The expansive strain or chemical prestress can be predicted with sufficient accuracy for practical purpose from the result of the standard expansion test, using hypothesis that the work done on the restraining steel bars by unit volume of expansive concrete is to be constant irrespective of concrete mix and curing[4]. Measured expansive strains with various steel percentages (p) were compared with the estimated values from the measured strains of standard specimens which had about 1 % of steel. The differences between the measured values and the estimated ones were within about 20 % as shown in Fig.4. The amount of expansive strain of reinforcing bars can easily become about 5×10^{-4} although it changes according to mix proportion of concrete, curing method, steel percentage and arrangement of bars. Therefore, crack widths of chemically prestressed concrete members could be greatly reduced compared with those in ordinary reinforced concrete members, because crack widths are proportional to the strain of tensile reinforcing bars.

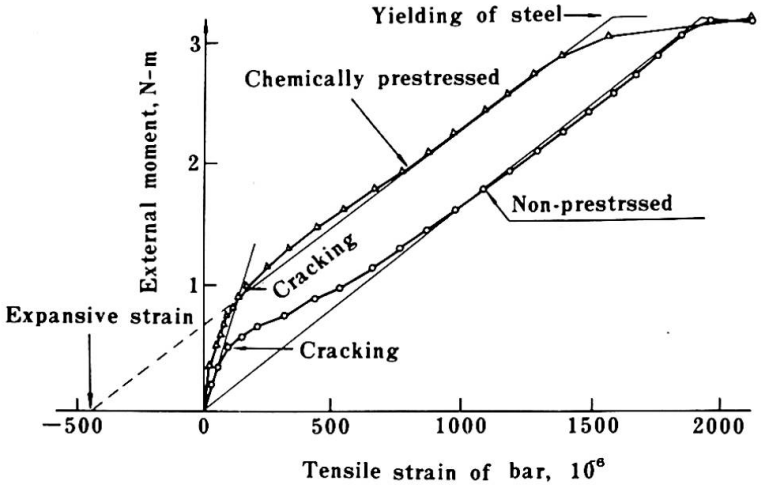

Fig.3 Increase in tensile strain of a bar in beam due to external moment

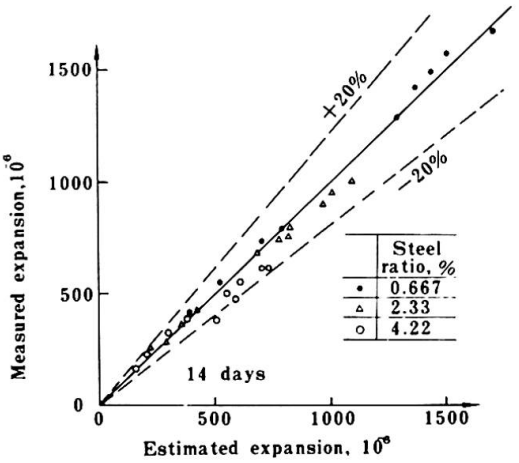

Fig.4 Measured and estimated expansions

2.2 Behavior of Frames

In order to ascertain the behavior of chemically prestressed concrete box culverts, experiments on frame specimens were conducted. The chemical prestress of beams of a frame specimen was about 3.0 N/mm^2, which coinsided with value in case of a different column rigidity and of the beam specimen with identical cross-sectional dimensions. For columns, when cross-sectional dimension was identical, roughly equal value was indicated, but when the cross sectional area of concrete was double leaving steel area equal, expansive strains of bars were increased by approximately 1.4 times while the chemical prestress was about 0.7 times. This suggests that the concept of "work done by expansive concrete" proposed by Tsuji[4] is also applicable to the case of frame.

Similarly to a beam specimen, flexural cracking resistance is improved due to chemical prestress, added to which increase in tensile strain of bars after cracking is smaller as shown in Fig.5. External moments in this figure were calculated from the applied loads on the upper beam of a specimen based on the elastic frame analysis assuming the flexural rigidities of the beams and columns to be constant. As the flexural rigidity of a beam is lowered after bending cracks are produced in it, the moments actually working will be smaller at beams and larger at columns.

After carrying out the tests on actual products, it was decided that the sections of slabs and walls in chemically prestressed concrete box culverts were reduced to 80 % of those in ordinary reinforced concrete box culverts. Besides reduction in quantity of materials, due to reduction of costs in transportation and installation accompanying lightening of product weights, concrete box culverts prestressed chemically are more economical than those without prestress.

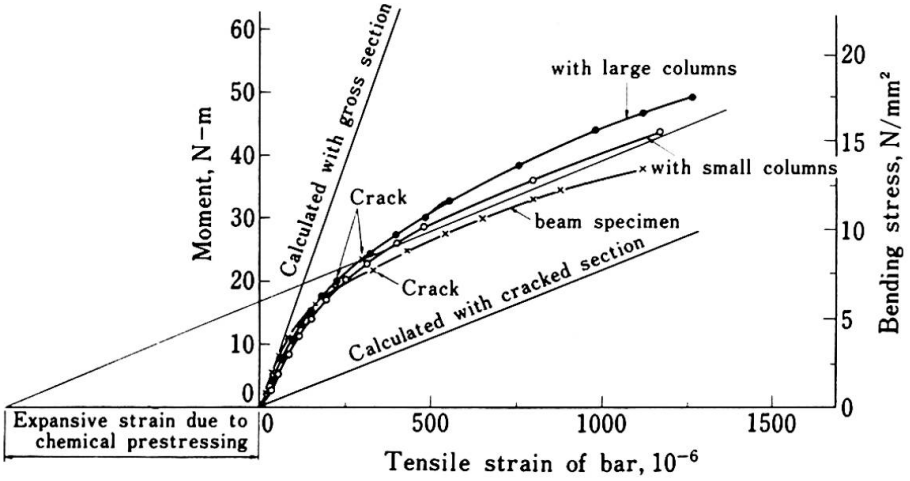

Fig.5 Increase in tensile strain of a bar in box culvert

3. Variation with Time in Effect of Chemical Prestress

For the purpose of confirming the existing of the effect of chemical prestress over a long period of time, experiments on the actual products were carried out. Specimens were selected from products representative of box culverts being sold on the market. Inner widths are 2,500 mm, inner heights 1,500 mm and specimen-lengths 900 mm. The cross sections of slabs and walls of specimens with chemical prestress were approximately 80 % of those without prestress. Two methods of storing specimens were adopted. One was a method reproducing actual conditions in which specimens were buried underground at the age of 2 months and excavated before testing. The other was a method of storing in air in a condition that specimens would not be exposed to rain assuming a state which would be the most adverse for effect of chemical prestress to continue to exist. Up to the present, loading tests have been completed for the ages of 14 days, 2 months and 9 months for storage in air, while testing at the age of 9 months has been completed for the case of underground storage. Further, there are six specimens still in storage and it is scheduled for final testing to be performed at the age of 5 years.

From the results of loading tests at each age, cracking moments were obtained as indicated in Fig.6. A cracking moment was calculated from the load when the first crack appeared in the upper slab, and the flexural cracking stresses in the figure were obtained from the cracking moments assuming concrete is an elastic material. The effect of chemical prestress at the age of 14 days was to increase the flexural cracking stress about 2.0 N/mm^2. When specimens were stored in air, there was recognized a slight reduction in cracking moment, but even at the age of 9 months the effect of chemical prestress was distinct. The cracking moment of chemically prestressed specimens buried underground from 2 months to 9 months were approximately 30 percent larger than the value at the age of 14 days. This result confirms the previous finding[2] that even if dried once the effect of chemical prestress is improved if members are placed in an atmosphere of high moisture.

4. Conclusions

(1) It is clarified that the behavior of box culverts subjected to bending moment can be greatly improved if chemical prestress is utilized.

(2) The effect of chemical prestress in precast concrete box culverts buried underground lasts for a long period of time. Even in case of products placed in air there still remained ample effect of chemical prestress.

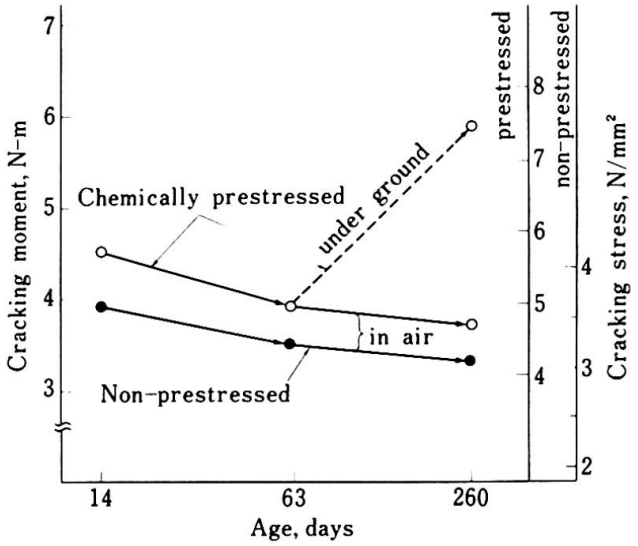

Fig.6 Cracking moments related to age of concrete

In essence, the application of chemical prestress to precast concrete box culverts is useful for cracking resistance and controlling crack widths over a long period of time. Besides reduction in quantity of materials, due to reduction of costs in transportation and installation accompanying lightening of product weights, concrete box culverts prestressed chemically can be more economical than those without prestress.

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SUMMARY

Flexural cracking resistance in a precast concrete box culvert is improved due to the effect of chemical prestress, added to which increase in tensile strain of reinforcing bars after cracking becomes smaller. The effect of chemical prestress lasts for a long period of time. Therefore, the sections of slabs and walls can be reduced to 80 % of those in usual culverts.

RESUME

La précontrainte chimique augmente la résistance des fissures à la flexion dans les éléments en béton préfabriqué de conduites souterraines; en outre elle diminue la contrainte de traction de l'armature après l'apparition des fissures. L'effet durable de cette précontrainte a pu être constaté, et l'on peut ainsi réduire la section des produits jusqu'à 80 % de la section normale.

ZUSAMMENFASSUNG

Chemische Vorspannung von vorgefertigten Kanälen mit kastenförmigem Querschnitt führt zu einer Erhöhung der Rissesicherheit und zu einer Beschränkung der Rissweite. Die Wirkung der chemischen Vorspannung bleibt lange erhalten. Aus diesem Grunde konnte die Dicke von Platten und Wänden auf 80 % des Ueblichen reduziert werden.

Reinforcing Bar Grouting Method for Connecting Precast Reinforced Concrete Structural Members

Méthode d'injection des armatures en acier pour la liaison d'éléments préfabriqués en béton armé

Injektion von Bewehrungs-Stäben für die Verbindung von vorgefertigten Stahlbeton-Bauteilen

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1. INTRODUCTION

"Reinforcing Bar Grouting Method" to be used for jointing and assembling the precast reinforced concrete structural members is shown schematically in Fig.1. The precast concrete elements are arranged the holes beforehand to be inserted the reinforcing bar. When assebling the elements, the re-bar are inserted into the corresponding holes. Then the cement mortar is grouted in the hole to make these elements into a composite unit. It does not need in-situ cast concrete.

This method may be applied to various types of connections making a good use of this method concept. Therefore the structural practicability of this method was experimentally examined. After that, this method was applied to several precast concrete structures.

This paper shows the results of the fundamental experiments.

2. BOND TEST

The bond strength of the re-bar fixed by grouting had been considered to control the structural character of the connection by this method. Therefore the pull-out bond tests, referring to the test method of "ASTM C324-57T", were carried out to know the significant factors affecting the bond strength.

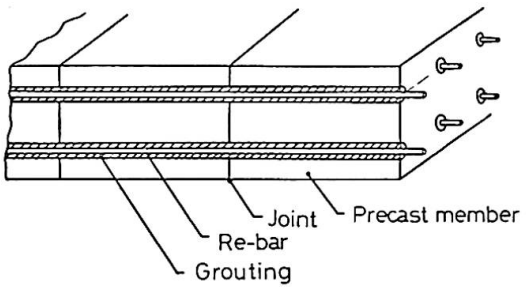
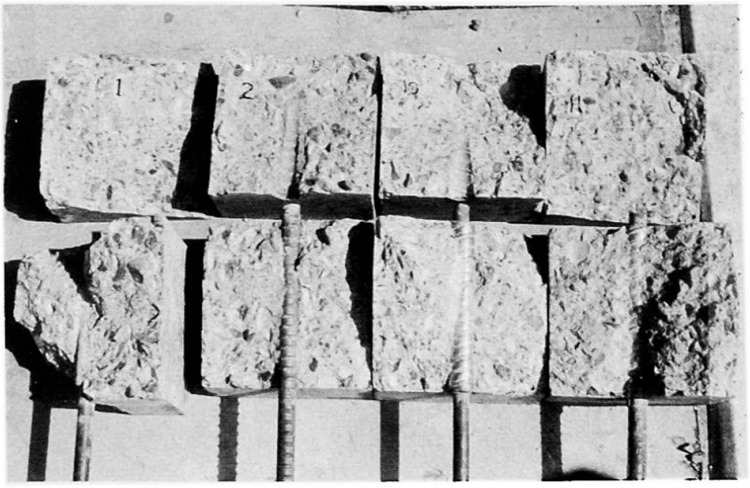


Fig.1 "Reinforcing Bar Grouting Method" concept

16 specimens made by grouting method were prepared under various conditions, i.e. using an aluminum powder admixture with or without expansive cement, making the hole with smooth surface or flexible sheath, grouting vertically or horizontally etc.. And 6 specimens with a directly embedded bar were also prepared for comparison.

The compressive strength of the cement mortal and the concrete used for specimens were about 270 kg/cm² and 335 kg/cm² respectively at the test, 7 days after grouting. The bond strength was measured in each specimen when the slipage of bar at the loaded side was 0.1 mm, 0.2 mm and the load reached to the maximum respectively. According to the variance analysis concerninig the effect of the abovementioned factors for these strengths, each factor had a slight influence. The bond strength of ϕ22 mm deformed bar fixed by grouting was about 80 kg/cm² (at 0.1 mm slipage) and 150 kg/cm² (max.) which were same with or more than that of a directly embedded bar. This confirms that the sufficient pull-out bond strength can be obtained using the conventional cement mortar grouting.



Phot.1 Inside of specimens after pull-out bond test

3. BENDING TEST

The re-bar in this method is generally used for connecting and it has possibility to be utilized as the main reinforcements in precast reinforced concrete structural members. Therefore the bending tests of beam were carried out to know the structural character of the joint and the member.

Dimensions, loading capacities and material characteristics of the beam specimens were as follows;

h=30cm, b=20cm, d=25cm, d'=5cm, $A_s/bd=A_s'/bd=0.011$,
L=200cm, l=150cm,
M_d=2.25t-m (for design calculated as σ_s=1800kg/cm²)
M_u=5.97t-m (for ultimate calculated as σ_s=3600kg/cm²)
where
$f_c \fallingdotseq 340$kg/cm² (for concrete when tested)
$f \fallingdotseq 240$kg/cm² (for mortar when tested)
f_y=3600kg/cm² (for ϕ19mm deformed bar yielding)

4 types of beam specimens, 2 specimens to each type, were prepared for testing.

(1) No-joint beam

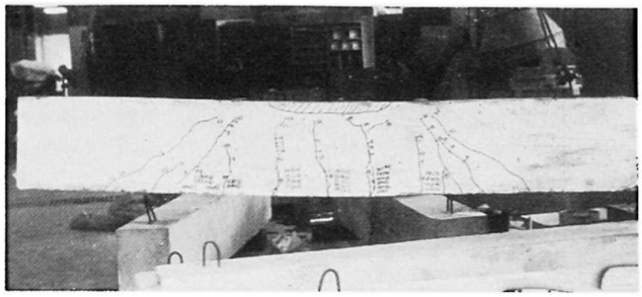
TYPE-A ; The no-joint beam specimen with the reinforcements fixed by grouting

TYPE-A' ; The conventional beam specimen with the directly embedded reinforcements (for comparison)

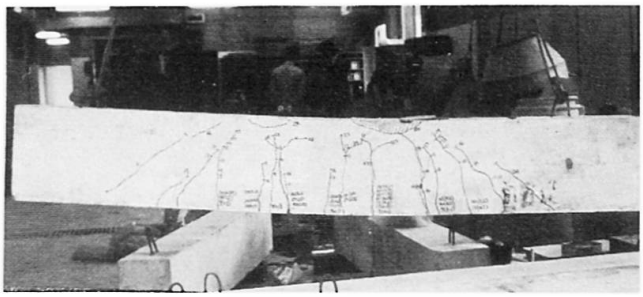
Between both types of specimens under same loading, there was no notable difference in crack pattern and crack width.(Phot.2 and Phot.3) This means that the bond strength of the re-bar fixed by grouting in beam under bending is almost same as that of the conventional beam. In other words, the bending character of the beam made by the grouting method can be insured as same as the conventional one.

(2) Beam with a joint

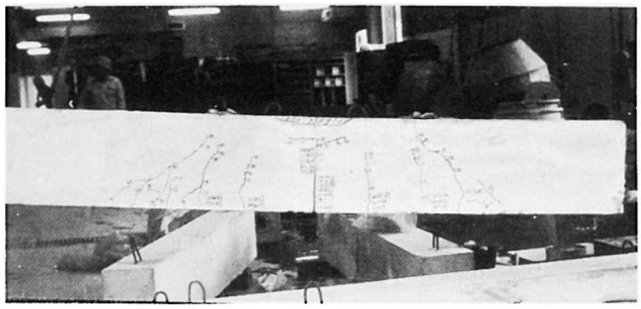
TYPE-B ; The precast beam specimen connected with two elements by the re-bar grouting method



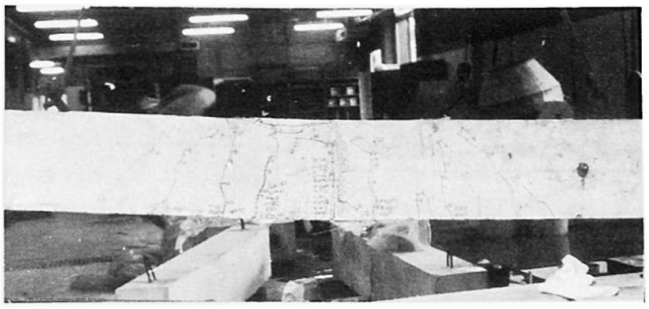
Phot.2 TYPE-A Specimen



Phot.3 TYPE-A' Specimen



Phot.4 TYPE-B Specimen



Phot.5 TYPE-B' Specimen

TYPE-B' ; The conventional beam specimen arranged a construction joint (for comparison)

In both types of specimens, the first crack occured at the joint, which developed as the most significant crack in same manner.(Phot.4 and Phot.5) The regidual deflection after removing the designed load was also similar. Therefore it was known that the re-bar grouting joint could be treated like a construction joint.

(3) Comparison of beam with and without joint

The degradation of flexural rigidity as bending moment increased is shown in Fig.2. The solid line (A) is for the TYPE-A specimen including the TYPE-A' specimen; the dotted line (B), TYPE-B including TYPE-B'.

The dotted line is lower in the range of small load than the solid line, however it is almost same at the designed load for the transformed section by the traditional reinforced concrete beam theory. And both lines show the same ultimate strength.

4. CONCLUSION REMARKS

Through the fundamental experiments, it was known that this method is available for practical use, on condition that the detailed checks depending on circumstances are added. This method may be applied to jointing and assembling the various precast reinforced concrete structural members. The method arranged by prestressing may also effective.

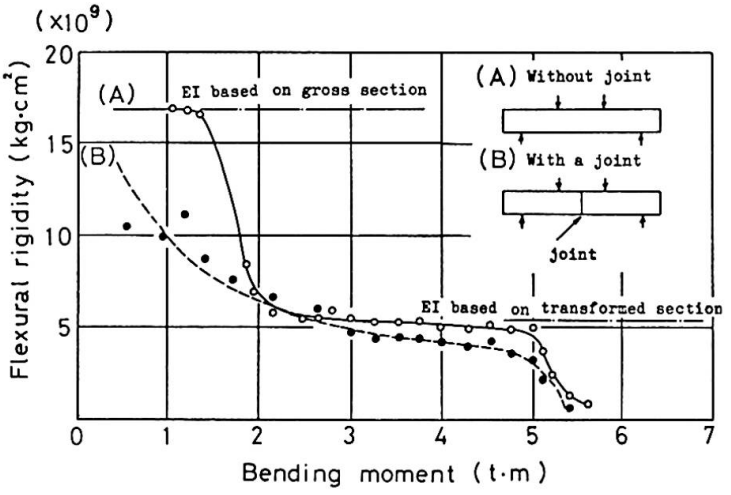


Fig.2 Flexural rigidity of beam with and without joint

SUMMARY

The reinforcing bar grouting method is used for connecting the precast reinforced concrete elements by means of grouted reinforcements. According to the pull-out bond tests and the bending test of beam, this method may be available for practical use depending on circumstances.

RESUME

Cette méthode d'injection des armatures en acier est utilisée pour la liaison d'éléments préfabriqués, en béton armé. D'après les essais d'adhérence par arrachement et les essais de flexion, cette méthode peut être utilisée en pratique, dans certaines circonstances.

ZUSAMMENFASSUNG

Die Injektion von durchgehenden Bewehrungs-Stäben für die Verbindung von vorgefertigten Stahlbeton-Bauteilen erweist sich aufgrund von Haftversuchen und einer Prüfung der Verbindung auf Biegung als geeignete Lösung.

VI

Béton léger de structure en préfabrication lourde

Leichtbeton für schwere Fertigteile

Lightweight Structural Concrete for Heavy Precast Construction

MAC N'GUYEN
Ing. Civil des Ponts et Chaussées

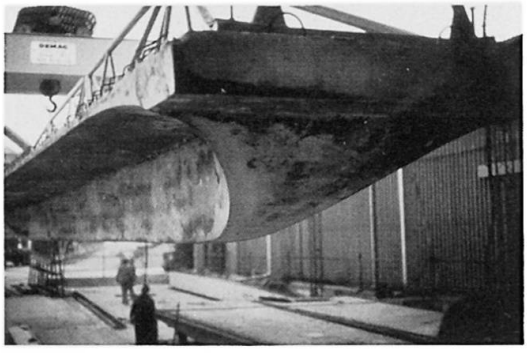
MICHEL AUPERIN
Ingénieur E.C.L.

Entreprise BOUYGUES
Clamart, France

L'Entreprise BOUYGUES a réalisé, pour la construction d'un immeuble de bureaux et d'habitation, 175 poutres préfabriquées en béton léger précontraint.

La géométrie de ces poutres est le résultat d'une recherche architecturale poussée, tant en ce qui concerne les lignes que l'optimisation des volumes. Leurs caractéristiques principales sont les suivantes :

- Section transversale nervurée de section quasi constante en partie centrale, la largeur de la nervure augmentant, et sa hauteur diminuant rapidement au voisinage des appuis,
- Hauteur maximum de 85 centimètres,
- Ames de 20 à 30 centimètres d'épaisseur en partie centrale,
- Table supérieure de 8 à 13 centimètres d'épaisseur, et de largeur variant de 2,95 à 1,88 mètres compte tenu du contour circulaire des planchers.



Vue d'une poutre après décoffrage

Les poutres de 16,37 mètres de longueur hors tout reposent isostatiquement sur des poutres par l'intermédiaire d'appuis en néoprène. Posées côte à côte avec un jeu de 1,6 centimètre, elles sont solidarisées par une chape rapportée, armée d'un treillis soudé, surépaissie au droit du joint. La surface totale du plancher ainsi constitué est de 7.000 mètres carrés.

Le béton léger utilisé ne comporte que du sable naturel (sable silicocalcaire de Basse Seine). Le granulat léger est un schiste expansé belge, l'Agral, dont la masse volumique en vrac est de 700 kg par mètre cube. Un rapport E/C assez élevé – puisqu'il est de 0,46 – et de l'utilisation de 2 kg par mètre cube d'un plastifiant réducteur d'eau ont permis d'obtenir un béton très maniable, le slump à la mise en place restant compris entre 6 et 10 centimètres.

La masse volumique du béton frais ainsi obtenu est comprise entre 1,8 et 1,9 tonnes par mètre cube, ce qui conduit à une masse volumique de calcul de 1,95 tonne par mètre cube, armatures comprises, si bien que le poids des poutres atteint 16 tonnes. La résistance minimale à la compression sur cylindre est de 45 Mégapascal, ou encore Newton par millimètre carré à 28 jours, et la résistance à la traction de 3 Mégapascal.

Les poutres sont précontraintes au moyen d'un câble de 12 torons de 13 millimètres. L'ancrage est du type Freyssinet trois pièces. La force maximale à l'ancrage est de 176 kdaN à la mise en tension.

Le calcul des poutres a été conduit en classe III des recommandations internationales.

Les poutres supportent des charges de cloison de 170 à 1.000 kg par mètre linéaire, et des surcharges réparties qui s'étagent de 600 à 1.200 kg par mètre carré, chape comprise. La contrainte caractéristique dans le béton atteint 20 Mégapascal à la mise en tension, et 5 Mégapascal en traction en service sous charges maximales, en fibre inférieure.

Les quantités mises en oeuvre ressortent, par mètre carré de plancher, à :

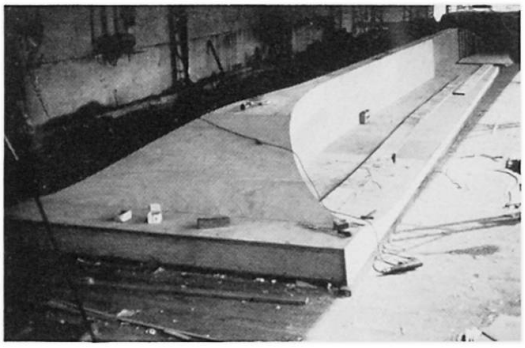
0,21 mètre cube de béton

3,8 kg d'acier de précontrainte, soit 18 kg par mètre cube

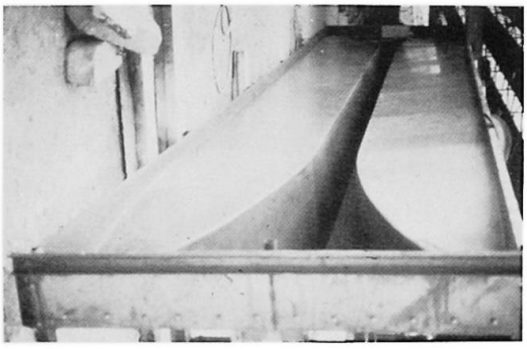
17,2 à 24 kg d'acier à haute adhérence de 420 Mégapascal de limite élastique, soit de 82 à 115 kg par mètre cube.

Les poutres ont été préfabriquées en usine. Le béton a été livré par une centrale de béton prêt à l'emploi ; les granulats légers, protégés par des bâches sur l'aire de stockage, n'ont pas été prémouillés avant malaxage. Le transport, dont la durée était de l'ordre de 30 mn, s'effectuait en camions malaxeurs.

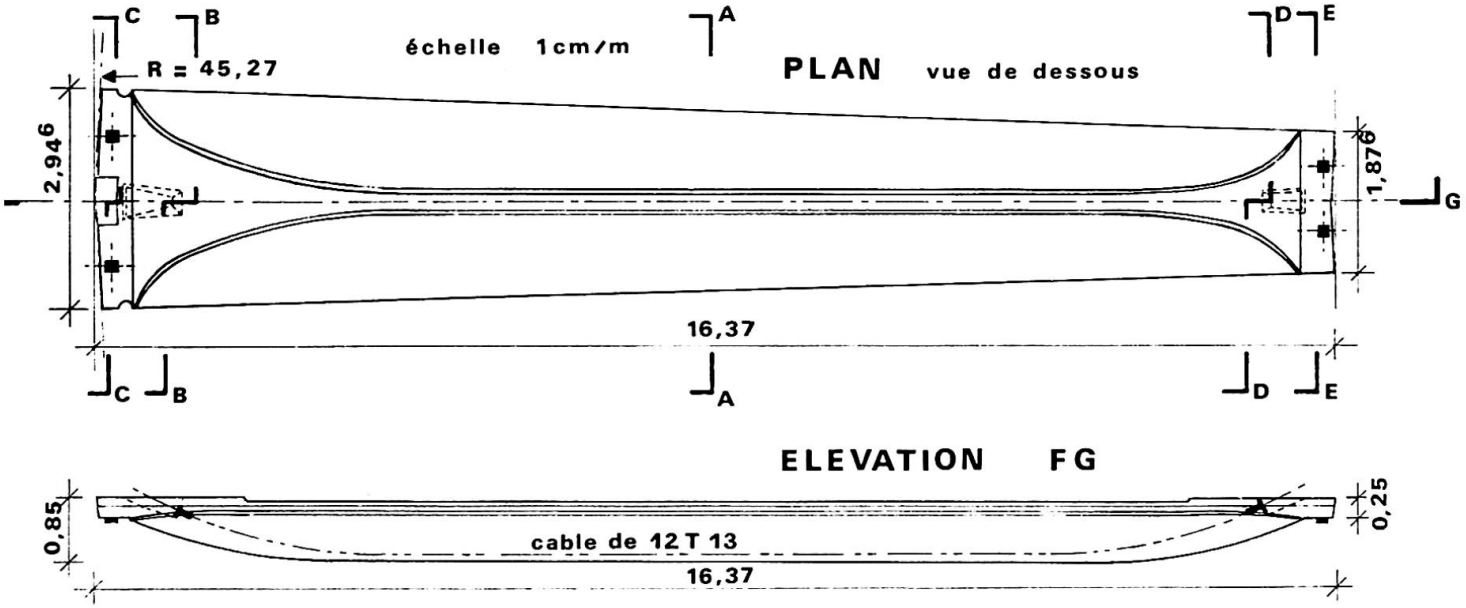
Compte tenu de la géométrie particulière des poutres, les 3 moules de coffrage ont été réalisés en polyester de 10 millimètres d'épaisseur, eux-mêmes moulés sur une matrice en bois.



Matrice du Moule

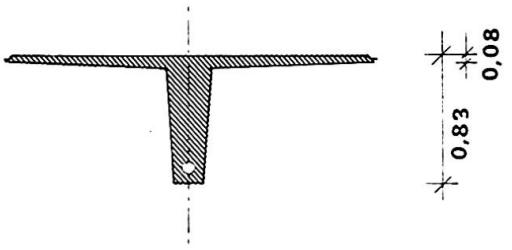


Moule en polyester

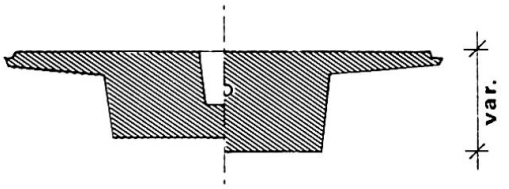


SECTION A A

échelle 2 cm/m



SECTION B B



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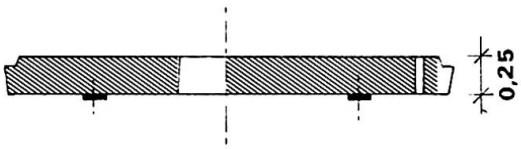

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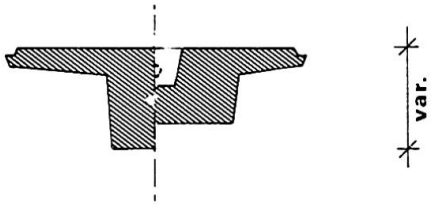
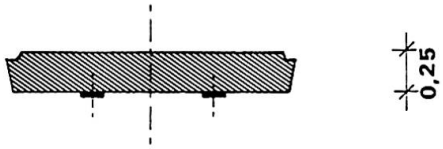
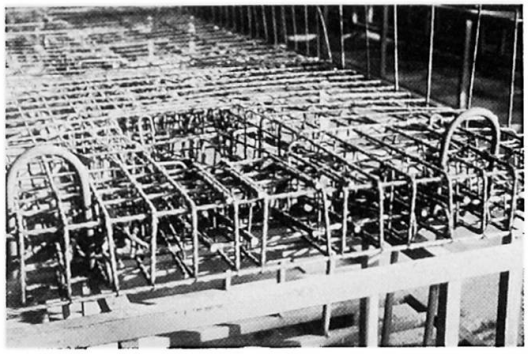


SECTION E E



Les armatures préfaçonnées ont été montées en 3 cages sur gabarits.



Vue partielle de la cage d'armatures

Le béton vibré avec des aiguilles de 50 millimètres était étuvé au moyen de résistances électriques noyées dans l'âme. La durée du cycle d'étuvage était de 10 heures, la température maximale de 55° centigrades. La durée du cycle de fabrication des poutres a été de 24 heures, leur mise en tension étant réalisée sur l'aire de stockage entre 3 et 7 jours.

La déformation des poutres à la mise en tension s'est révélée conforme à la prévision, donnant des contreflèches de 2 centimètres environ. Des essais de chargement effectués sur 4 poutres ont donné des flèches comprises entre 26 et 32 millimètres, pour 30 prévus, confirmant la valeur du module sécant d'élasticité instantané, évalué à 20.000 Mégapascal.

Il est a noter qu'aucune fissuration n'a été constatée au voisinage des pièces d'ancrage, directement appuyées sur le béton léger.

En conclusion, la préfabrication de poutres en béton léger ne semble pas poser de problèmes particulièrement délicats, pas plus que pour celle en béton traditionnel.

Les expériences acquises en usine dans la fabrication du béton léger et dans sa mise en oeuvre, ainsi que celles obtenues par les bureaux d'études dans l'approche des phénomènes de résistance et de déformation, portent aujourd'hui la promesse d'une utilisation plus courante du béton léger.

RESUME

L'entreprise a réalisé 175 poutres préfabriquées en béton léger précontraint. L'utilisation de granulats de schiste expansé a permis d'obtenir une résistance à la compression sur cylindre comprise entre 45 et 52 Mégapascal. Aucune fissuration n'a été constatée au voisinage des pièces d'ancrage, directement appuyées sur le béton léger.

ZUSAMMENFASSUNG

Die Firma hat 175 vorgespannte Betonträger aus Leichtbeton hergestellt. Die Zylinder-Druckfestigkeit beträgt zwischen 450 und 520 kp/cm^2 wenn der Leichtzuschlag aus ausgebreitetem Schist besteht. In der unmittelbaren Umgebung der Vorspannungsverankerung wurden keine Risse beobachtet.

SUMMARY

The company has manufactured 175 precast posttensioned beams made of lightweight aggregate. The compressive strength obtained after 28 days on cylindrical testing samples using expanded shist-aggregates varies between 450 to 520 kg/cm^2. No cracking has been descrued in the immediate vicinity of the anchorages which bear directly on the light weight concrete.

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VI

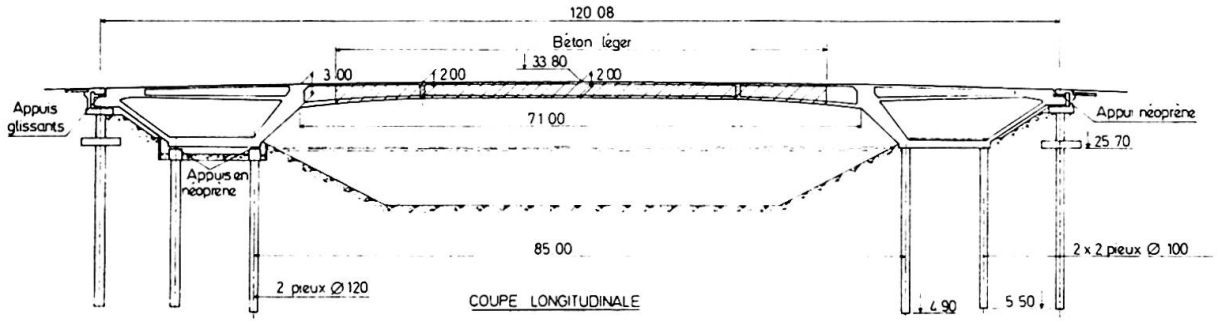
Idées nouvelles pour la construction de ponts en béton léger

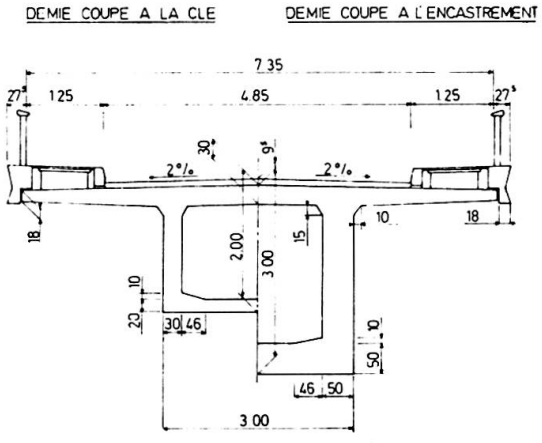
Anwendung des Leichtbetons im Brückenbau

Use of Light Concrete in Bridge Engineering

MICHEL VIRLOGEUX
Ingénieur des Ponts et Chaussées
S.E.T.R.A.
Bagneux, France

Ce n'est que récemment que l'on a commencé, en France, à s'intéresser au béton léger, du fait de l'abondance de matériaux traditionnels de bonne qualité. Le bulletin de 1975 de l'Association Française des Ponts et Charpentes (A.F.P.C.) donne une description assez détaillée des premiers ponts qui ont été construits. Bien que ces ouvrages soient encore modestes par rapport à certaines réalisations étrangères, notamment allemandes et néerlandaises, on peut citer le pont de Bruyères-sur-Oise, qui a été projeté par la STRUCTEC. Il s'agit d'un pont formé de deux portiques en béton traditionnel, sur lesquels s'encastre une travée centrale de 71 mètres en béton léger.

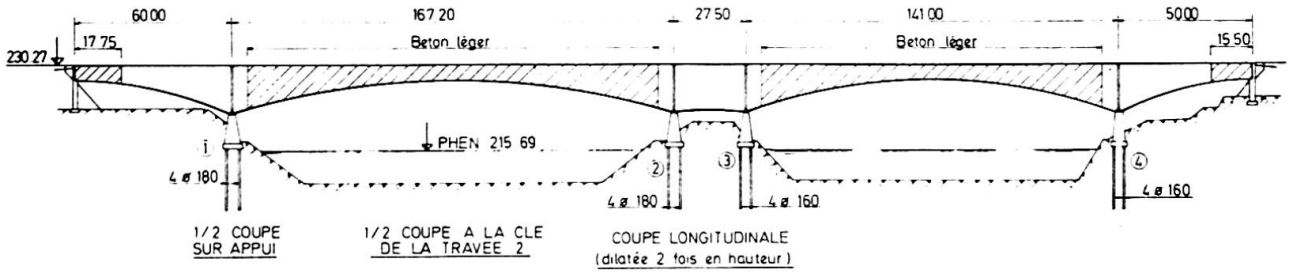




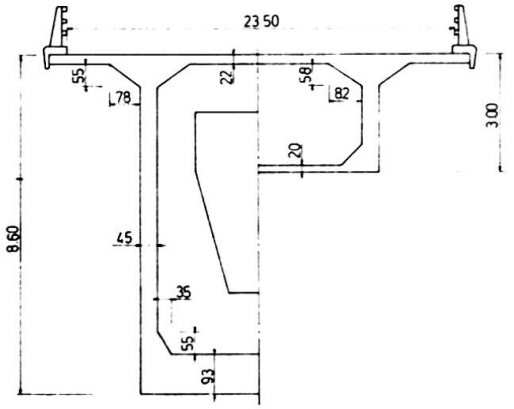
Pont de Bruyères-sur-Oise

Elle a été construite sur cintre, par voussoirs, la dérivation de l'Oise ayant été creusée une fois le pont terminé. Le terrain ne permettant pas de reprendre facilement des poussées, les déplacements horizontaux ont été libérés d'un côté, en interposant entre les pieux et le radier des appareils d'appui glissants.

Des projets plus ambitieux ont été établis récemment, notamment par le S.E.T.R.A., pour le franchissement du canal d'Alsace, à l'aval de l'usine hydroélectrique d'Ottmarsheim. L'emploi du béton léger dans les deux grandes travées permet de réduire les dimensions des fondations, d'amincir

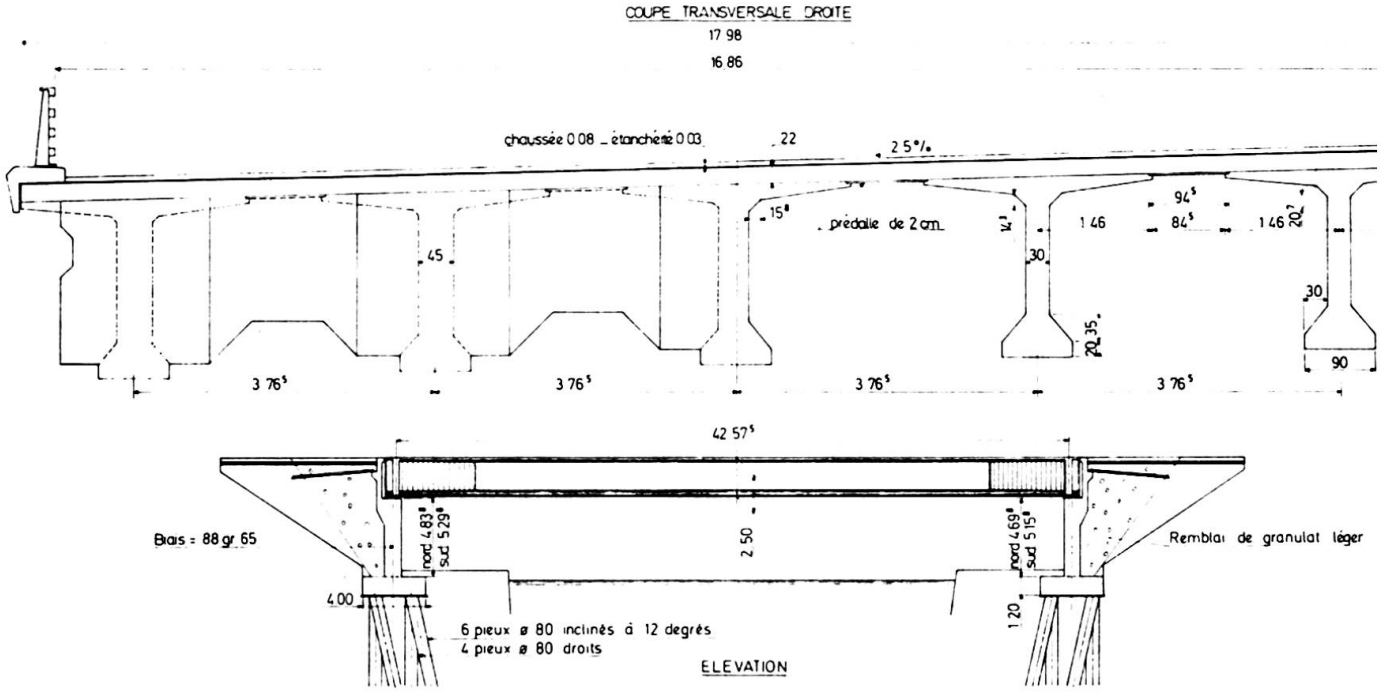


Projet de pont d'Ottmarsheim.



légèrement le tablier, et, par voie de conséquence, de diminuer de quelques mètres les portées des grandes travées. Mais surtout, le maintien du béton traditionnel dans les travées de rive permet, par un effet de contrepoids maintenant classique, de les raccourcir et de les alléger. D'autres solutions ont été étudiées, notamment une solution à deux travées haubannées en béton précontraint. L'appel d'offres est en cours.

Dans les années à venir, l'emploi du béton léger devrait se développer dans le domaine de la préfabrication. En effet, l'allègement qu'il apporte permet des gains sur les moyens d'exécution, qu'il

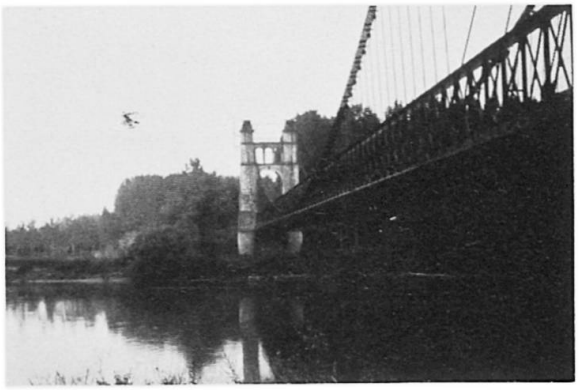


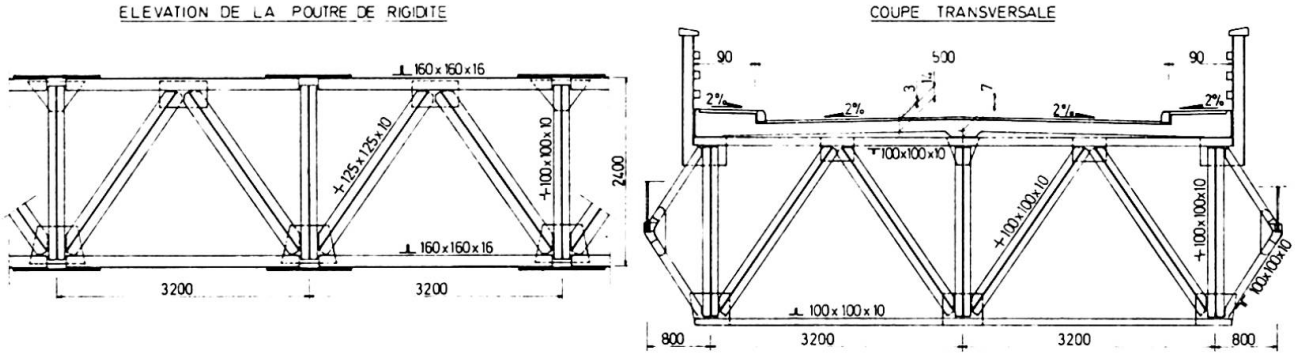
Pont de Calais.

s'agisse des cintres (et on peut penser, en particulier, aux cintres autolanceurs) ou, surtout des engins de manutention de pièces préfabriquées. C'est ainsi que l'emploi du béton léger a permis de réduire de 150 à 110 tonnes le poids des poutres préfabriquées du pont de Calais, en cours de construction. A l'inverse, pour un matériel donné, l'emploi du béton léger permet d'augmenter la taille des élément préfabriqués, poutres ou voussoirs.

Mais le béton léger devrait surtout permettre d'imaginer des structures nouvelles, ou de rendre compétitive des structures qui ne l'étaient plus en béton traditionnel.

Tout d'abord, une solution originale, étudiée par le C.E.T.E. de Lyon et le Centre Technique de l'Aluminium, a été retenue pour la reconstruction du tablier du pont de Groslée, qui comporte une travée unique de 174 mètres sur le Rhône. Le tablier, qui forme poutre de rigidité, est une ossature mixte, aluminium - béton léger. L'association de ces deux matériaux permet de limiter le poids total du tablier à 600 kg/m^2 étanchéité et couche de roulement comprises, soit un gain de l'ordre de 250 kg/m^2 par rapport à une solution classique acier - béton traditionnel.





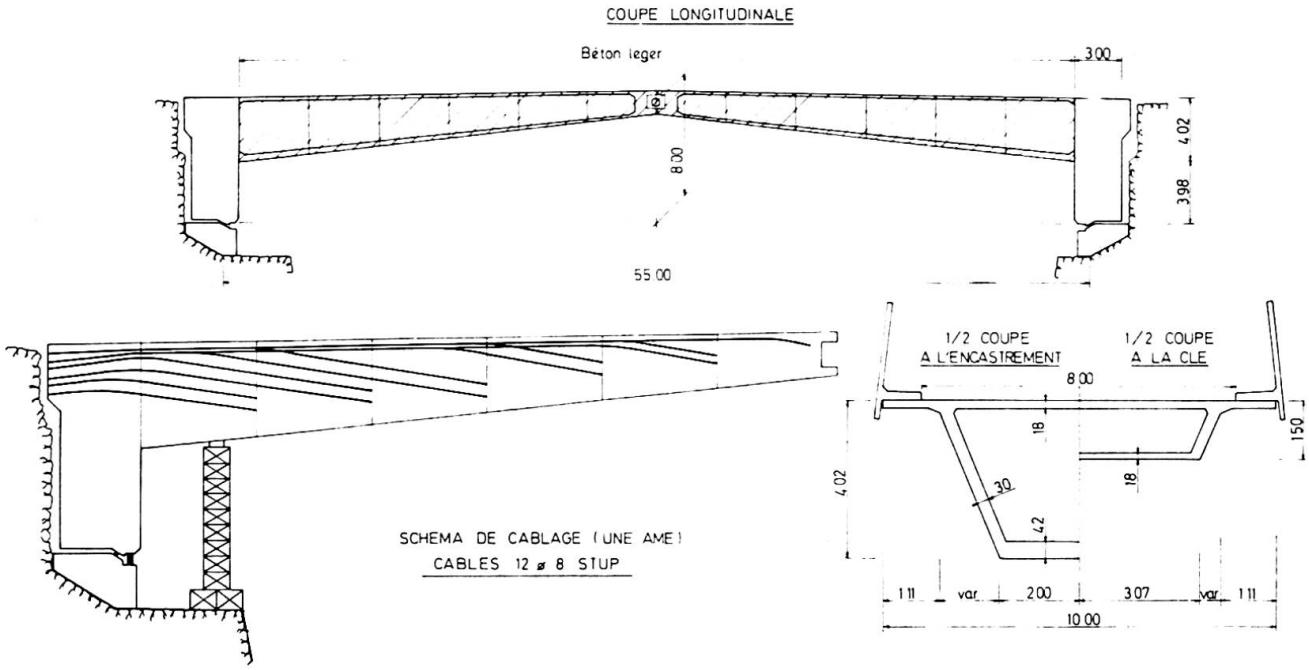
Pont de Groslée.

Une autre idée, qui devrait bientôt faire l'objet de recherches expérimentales, consiste à utiliser le béton léger pour la construction de dalles de couverture de type Robinson. Ces dalles sont formées d'une tôle de 8 à 12 millimètres, munie de connecteurs la liant au béton dont l'épaisseur est de l'ordre de 10 centimètres. Le poids serait d'environ 270 kg/m^2, en choisissant un béton très léger, soit un gain d'environ 80 kg/m^2 par rapport à la dalle Robinson classique. L'emploi du béton léger devrait aussi limiter le nombre des raidisseurs. Ces chiffres sont à rapprocher de ceux qui sont donnés par Messieurs Toshikazu Suruga et Yukio Maeda, dans le rapport préliminaire. Cela conduirait cependant à des structures de conception assez différente de celles que l'on projette avec une dalle orthotrope.

Enfin le béton léger doit permettre, dans certains cas, de réaliser économiquement des arcs par encorbellements successifs.

Un arc à trois articulations vient d'être projeté par la S.E.T.E.C. et le S.E.T.R.A., pour une passerelle à gibier de 56 mètres de portée, dans les Vosges. L'emploi du béton léger a permis de réduire

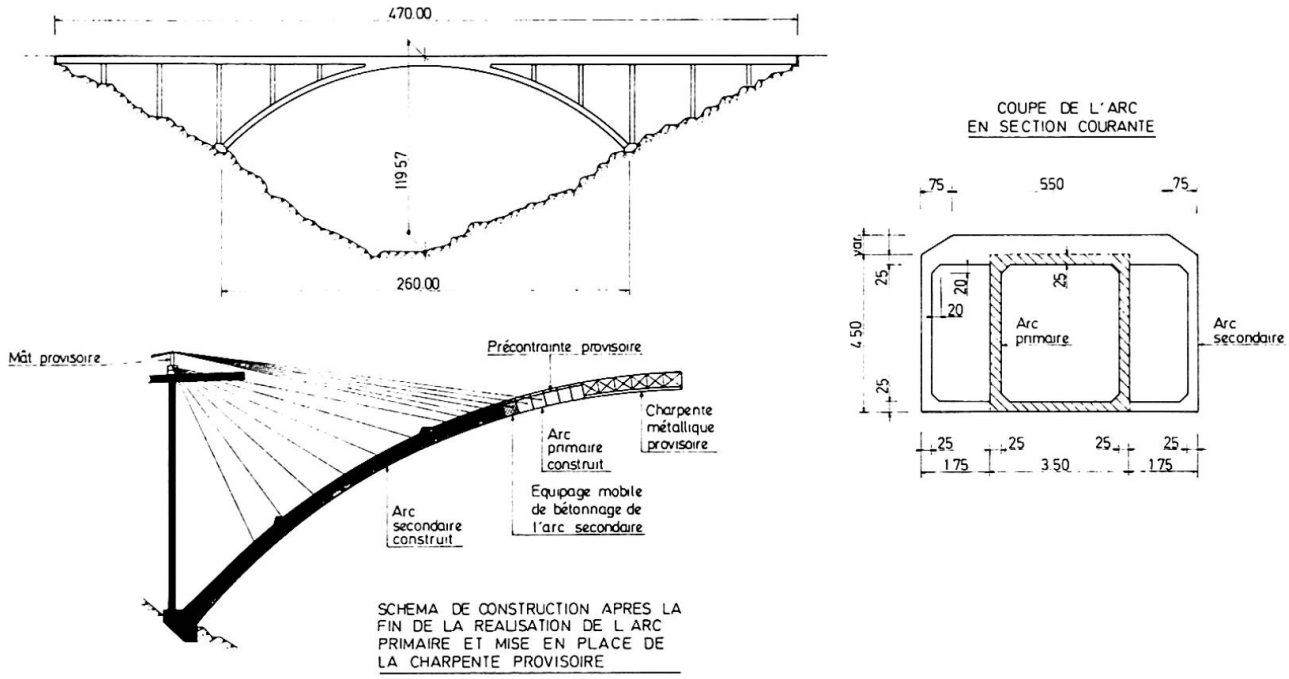
les moments de déséquilibre en phase de construction, ce qui limite le contre-poids ou les ancrages dans le rocher. La poussée de l'arc est telle que le câblage de construction des fléaux est surabondant en service. Cette solution n'a cependant pas été retenue, car une solution en bois lamellé-collé s'est avérée encore plus économique dans ce cas particulier.



Projet de la passerelle de Saverne.

Un des viaducs de Caracas.

En poursuivant cette idée, on arrive aux arcs construits par encorbellements successifs avec haubannage provisoire, dont un exemple est donné par Monsieur Miyazaki dans le rapport préliminaire. Le principe du haubannage provisoire est ancien puisqu'il a déjà été utilisé par Freyssinet pour la construction des viaducs de Caracas. L'emploi du béton permet de diminuer de 30 % la section des haubans de construction, et de presqu' autant les efforts en service. On voit tout l'intérêt que cela peut présenter pour les très grandes portées. Pour s'en faire une idée, on peut se référer à la solution en arc qui avait été proposée par l'Entreprise Campenon-Bernard, en béton traditionnel pour le viaduc du Magnan (une solution classique avait finalement été préférée, à égalité de prix).



Projet d'arc pour le viaduc du Magnan.

Il n'est pas possible d'apporter de conclusion générale, mais, dans le domaine du béton précontraint, on peut quand même insister sur l'intérêt que présente le béton léger lorsque l'ouvrage présente de fortes dissymétries, définitives ou de construction.

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RESUME

L'emploi du béton léger peut permettre des économies dans le domaine de la préfabrication. Mais il peut aussi rendre compétitives des solutions non classiques ou nouvelles, soit par leur grande légèreté (dalle Robinson, association avec l'aluminium), soit par la réduction d'importants efforts de dissymétrie (construction par encorbellements dissimétriques).

ZUSAMMENFASSUNG

Die Anwendung von Leichtbeton führt zu Einsparungen bei der Vorfertigung von Bauwerken. Auch neuartige Lösungen werden dank des leichteren Gewichtes wirtschaftlich (Robinson-Platten, Verwendung von Aluminium). Insbesondere bei unsymmetrischen Tragwerken (Freivorbau) zeigen sich Vorteile.

SUMMARY

Using lightweight concrete may lead to saving in precast structures. But it may also make non conventional or new solutions competitive, either due to their extreme lightness (Robinson type slabs, lightweight concrete - aluminium composite structures), or due to reduction of important dissymmetry forces (construction by cantilever method for dissymmetrical bridges).

VI

Continuité des poutres préfabriquées précontraintes par adhérence

Durchlaufwirkung von im Spannbett vorgespannten vorgefertigten Trägern

Continuous Multiple Span Beam by Connection of Precast Prestressed Elements

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A l'occasion de la construction d'un viaduc longeant la rivière LA GARONNE dans l'agglomération de BORDEAUX, en 1974, nous nous sommes posé le problème de la conception et du calcul d'une structure constituée par des poutres préfabriquées en béton précontraint par armatures adhérentes, rendues continues sur appuis par coulage en place d'un chevêtre en béton armé. Le viaduc de cette voie sur berge comprend en effet 131 travées de 13,50 m de longueur chacune. Ces travées sont solidarisées entre elles et à leurs appuis (piles-pieux) de manière à constituer une succession de portiques encastrés à 5 travées.

Au cours de l'année 1974, nous avons constitué 2 montages représentatifs de la structure, afin de les soumettre à des essais de chargement. Le premier montage a subi l'essai de chargement 20 jours après la réalisation du clavage en béton armé. Les conditions et les résultats de cet essai sont relatés dans le rapport préliminaire, page 568 et suivantes.

Le deuxième montage, sensiblement identique au précédent, a été réalisé 21 jours après la fabrication des poutres et a été suivi pendant 18 mois, avant de subir finalement un essai de chargement jusqu'à la rupture, le 2 Juin 1976.

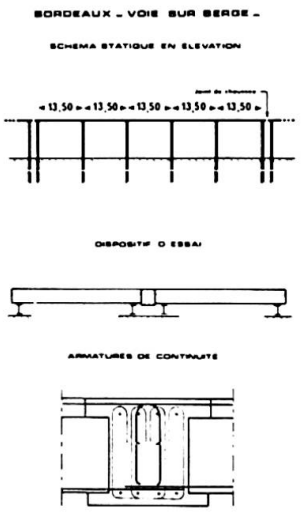


Fig. 1 Schéma statique et dispositif d'essai

1 — Suivi du montage

L'objet des mesures faites pendant la période de 18 mois qui a suivi la réalisation du second montage était de suivre l'évolution des redistributions d'efforts dus au fluage des poutres précontraintes préfabriquées, gênées dans leurs libres rotations par le chevêtre en béton armé. Nous avons pour cela procédé à la pesée des réactions d'appuis à l'aide de pesons à lame de flexion posés dès la réalisation du clavage et laissés en permanence sous les appareils d'appuis.

Afin de pouvoir dissocier les effets du fluage de ceux des gradients thermiques, nous avons implanté des sondes de température en divers points de la structure.

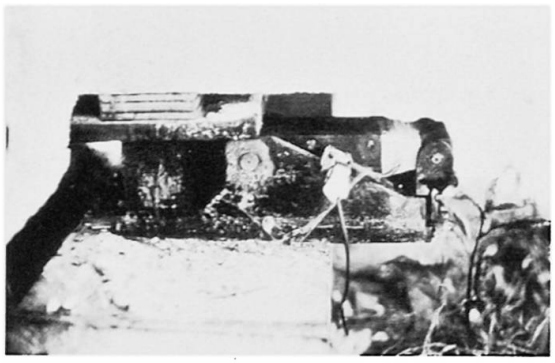


Fig. 2 Pesée permanente des réactions d'appuis.

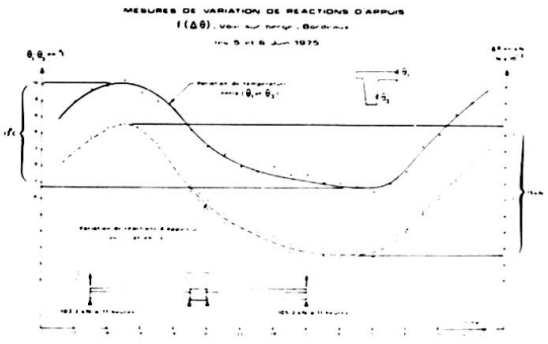


Fig. 3 Corrélation réaction d'appui / gradient thermique.

Le suivi à long terme de notre modèle a abouti aux résultats suivants :

a) Les variations journalières des réactions d'appuis constatées à l'issue de différents cycles de mesures ont été relativement importantes comme l'indique ce dessin qui récapitule les résultats d'une journée ensoleillée. Les températures mesurées au coeur du béton ne suivent pas une loi linéaire sur la section. Nous avons pu constater cependant que les redistributions d'efforts mesurées correspondent sensiblement à des différences fictives de température variant entre 8°C et – 2°C entre fibres extrêmes d'une même section, calculées en supposant un gradient thermique linéaire.

b) Les redistributions d'efforts dues au fluage des poutres et aux pertes de précontrainte ont été évaluées à partir des mesures de réactions d'appuis et compte tenu de la correction précédente. Les résultats révèlent une évolution de ces réactions, que nous représentons ici pour un appui d'extrémité : On constate en particulier une augmentation de 22 % de la réaction sur cet appui au cours des 18 mois qui on suivi le clavage.

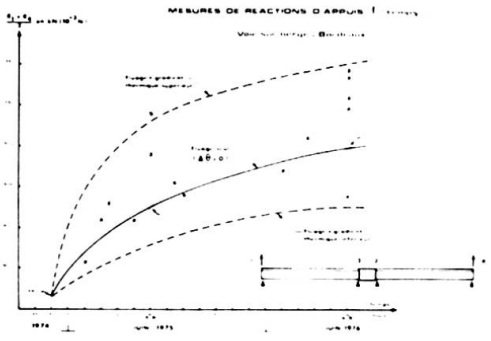


Fig. 4 Evolution des réactions d'appuis.

Les mesures nous permettent de conclure en l'existence d'un moment fléchissant positif permanent sur les appuis de continuité avec mise en traction de la fibre inférieure (1). Nous sommes donc à l'opposé des structures analogues coulées en place. Ce fait doit être particulièrement pris en considération par les projeteurs afin qu'ils sachent éviter les désordres consécutifs à une sous-estimation des armatures passives en fibre inférieure au voisinage des appuis.

2 — Essais de chargement

L'essai de chargement s'est déroulé le 2 Juin 1976 et il a permis d'exécuter différents cycles de chargement d'intensité croissante. Parmi les nombreux résultats recueillis, trois peuvent retenir plus particulièrement notre attention :

a) Le diagramme liant la réaction R sur un appui d'extrémité à la charge F appliquée dans chaque travée, révèle le fonctionnement hyperstatique de la structure ainsi que le transfert de charge accompagnant l'apparition d'une rotule plastique.

b) Le diagramme R/F en fonction de F précise d'ailleurs le processus de ruine qui s'amorce à partir de la section d'appui (cycle 2) et s'achève par la section à mi-portée (cycle 3). Les diagrammes théoriques se réfèrent à des matériaux élastoplastiques parfaits.

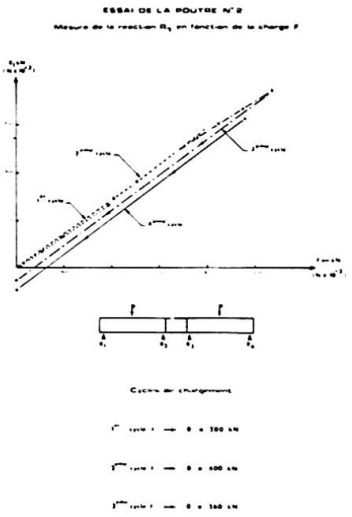


Fig. 5 Relation réaction/charge.

c) Enfin, nous avons indiqué sur ce diagramme les valeurs de R.F obtenues par voie théorique en supposant la section d'appui non fissurée (droite 1) et ensuite en supposant que tout le béton tendu de cette section est fissuré (droite 2) : nous constatons un cheminement de la droite 1 vers la droite 2 lors des essais répétés. Cette évolution semble révéler l'incidence de la fissuration du chevêtre, fissuration qui atténue la liaison hyperstatique de la structure. On rejoint ici les constatations relatées par MM. OKADA, OHURA et HOSAKA dans le rapport préliminaire.

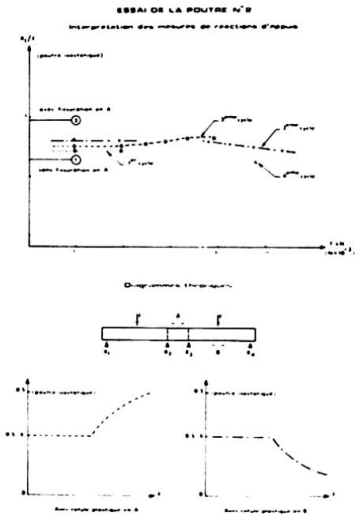
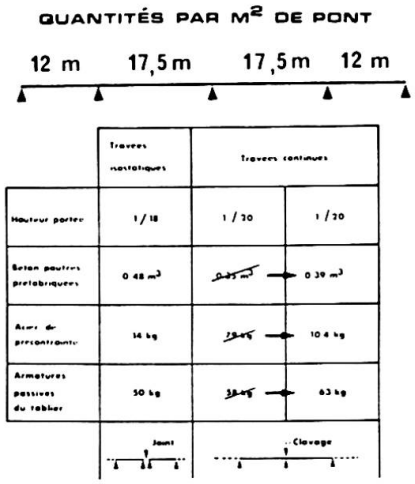


Fig. 6 Relation R/F/ en fonction de la charge F appliquée.

(1) La liaison entre travées étant assurée par un chevêtre en béton armé, le moment dû aux forces de précontrainte est nul dans ces sections,aux pertes de tension près (terme hyperstatique). De plus, retrait et fluage des poutres mettent en traction les zones de clavage.

Les conclusions de ces mesures et de ces essais ont été appliquées au calcul d'ouvrages utilisant cette technique — Sur la base du calcul d'un ouvrage à 4 travées dont les portées sont respectivement de 12 mètres, 2 x 17,50 mètres et 12 mètres, nous avons établi la comparaison suivante (ph.9).



– pont à 4 travées indépendantes ;

– pont à 4 travées rendues continues en ignorant les efforts supplémentaires étudiés. Ce sont les résultats rayés figurant sur le tableau ;

– pont à 4 travées rendues continues et calculé en tenant compte de ces efforts supplémentaires.

Fig. 7 Exemples de dimensionnement.

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RESUME

L'auteur présente les résultats des mesures faites sur des poutres préfabriquées rendues continues sur appuis par un chevêtre en béton armé, coulé en seconde phase. Le montage réalisé aux fins d'essais a été suivi pendant 18 mois. Les redistributions d'efforts dues au fluage des poutres précontraintes ont pu être évaluées après correction tenant compte de l'influence des gradients de température. Enfin , les essais de rupture pratiqués en juin 1976 sur ce montage sont présentés et reçoivent ici une tentative d'interprétation.
Les répercussions de cette étude sur le dimensionnement des ouvrages relevant de cette technique sont examinées au dernier alinéa.

ZUSAMMENFASSUNG

Die Resultate von Messungen an vorgefertigten, nachträglich zu einem Durchlaufträger verbundenen Balken werden mitgeteilt. Die Schnittkraftumlagerungen infolge Kriechen des Betons wurden während 18 Monaten verfolgt. Sodann fand ein Bruchversuch statt.
Schlussfolgerungen in Bezug auf die Bemessung derartiger Konstruktionen finden sich im letzten Abschnitt.

SUMMARY

This paper presents the results of some measurements on structures made up with precast members assembled into statically indeterminate structures by the pouring of concrete in a second phase. The effects of the creep of the prestressed concrete have been measured during a period of 18 months after the construction of the connexion. The results are discussed after corrections due to thermal gradients. The second part deals with the loading test conducted in order to check the long-term hyperstatic behaviour of the structure.
Consequence of these studies on the design of continuous structures made up with prefabricated elements are presented in the last part of this paper.

Precast Prestressed Concrete Truss Railway Bridge using Extremely High Strength Concrete

Pont ferroviaire en treillis en béton préfabriqué et précontraint utilisant du béton de très grande résistance

Eine Eisenbahn-Fachwerkbrücke aus vorfabrizierten vorgespannten Bauteilen aus höchstfestem Beton

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1. Introduction

The Akkagawa railway bridge, completed in 1975 by the Japan Railway Construction Public Corporation, is the first precast, prestressed, concrete truss bridge to be erected in Japan and is also the first structure in which autoclave cured, extremely high strength concrete has been used.

2. Planning of the Truss Bridge

The Akkagawa bridge was planned as part of the construction of a new railway line. An Akkagawa Bridge Committe was set up and the planning and design of the bridge discussed thoroughly by experts from each field.

In selecting the type of structure for the bridge, main consideration was given to the following three points:

(1) The structure should incorporate aspects of the most recent technical developments in bridge design.

(2) In view of its proximity to the sea coast, the bridge should be a concrete structure because of its advantage of requiring almost no maintenance.

(3) Since the bridge was to be located in a scenic, steep-sided valley, the structure should reflect aesthetic considerations.

With these considerations in mind, various structures were proposed and after detailed examination by the Bridge Committee, the PC truss structure was selected. It was also decided that autoclave cured concrete with a compressive strength of 800 kg/cm^2 should be used in the PC truss structure.

3. Preliminary Testing

Prior to the construction of the truss bridge, a series of preliminary tests were conducted to determine and iron out any problems in the materials, autoclave curing, design, erection, etc. The following were the main tests carried out.

3-1 Compressive Fatigue Test on Extremely High Strength Concrete

Because truss members are subject to repeated axial stress, the behaviour of compressive fatigue was investigated using autoclave cured concrete with a compressive strength of more than 800 kg/cm^2.

The tests showed that the behaviour of compressive fatigue in members made of extremely high strength concrete differed little from that in members made of ordinary strength concrete and it was found that the compressive fatigue strength after one million repeated loads was 55-60% of the static compressive strength.

3-2 Autoclave Curing of Large Section Members

The representative section of the truss members was 55 x 55 cm and to determine a suitable cycle for the curing of such massive concrete members, a thermocouple was used to investigate the internal temperature distribution. Through these tests, the optimum temperatures and time for the curing cycle were determined.

3-3 Photoelasticity Test and Finite Element Method Stress Analysis

Photoelasticity tests were carried out to examine the degree and flow of secondary stress which occurs in a truss structure as a whole through differences in panel point dimensions and joining methods and the results were incorporated in the design data.

A stress analysis using the finite element method was made on the panel points and the results used in determiming the measurements and form of the panel points.

3-4 Model Test

Using a four span, 1:5 scale model , as shown in Fig. 1, static tests were conducted to investigate the flexural rigidity, stress, crack development and type of fracture which occur in the truss structure. Dynamic tests were also carried out to study fatigue behaviour.

The test results agreed very closely with the calculated values and confirmed the suitability of the PC truss structure.

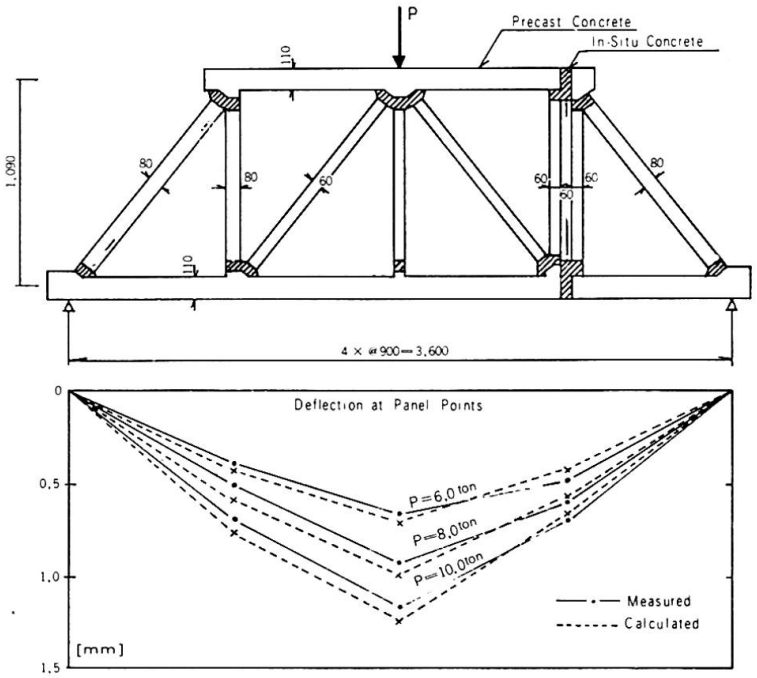

Fig. 1 Deflection Determined From Load Tests on 1:5 Model.

3-5 Load and Vibration Tests on a 24 Meter Span Bridge

Before construction of the Akkagawa bridge, a bridge of identical design, but with a single span of 24 meters, corresponding to a scale of 1:2, was erected at Otanabe and as well as studying problems arising from the design and erection of the bridge, manufacture of the members, etc., load and vibration tests were carried out and the safety of the bridge confirmed.

The ratio of measured stress to calculated stress when a 72 ton load was applied was between 0.80 and 0.96 and the relatively close agreement of these values was confirmed.

The vibration tests were conducted by setting up a 15 ton duplex vibration generator in the middle of the span. The peak of the resonance curve appeared at 7 Hz vertically and at 4-6 Hz horizontally, confirming the safety of the design against earthquakes.

4. Design

The Akkagawa railway bridge is a precast, prestressed concrete truss bridge with an overall length of 305 meters, consisting of six 45 meter spans and one 27 meter span, as shown in Fig. 2

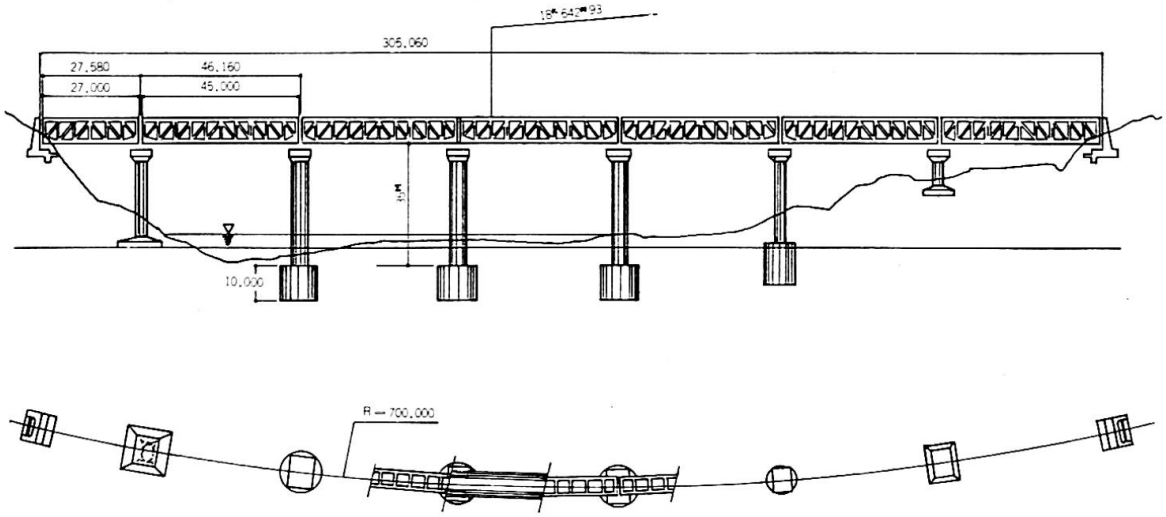

Fig. 2 Side View and Plan of PC Truss Bridge

The Howe truss was chosen because of the simplicity of manufacture of its members, assembly and erection and because of its suitability as a concrete structure. Consideration of the problems associated with the foundations and high piers led to the adoption of the deck bridge structure.

Fig. 3 shows the dimensions of a 45 meter span made up of 10 panels, each 4.5 meters long. The representative section of the upper and lower chord members is 55 x 55 cm and that of the diagonal and verticle members is 30 x 55 and 40 x 55 cm. A tension of 800 tons (280 kg/cm^2 prestress) was introduced in the eight cables in the lower chords, 100 tons in the two cables in the upper chords and a suitable degree of prestress was introduced in the verticals. However, no prestress was introduced in the diagonals, these being compression members.

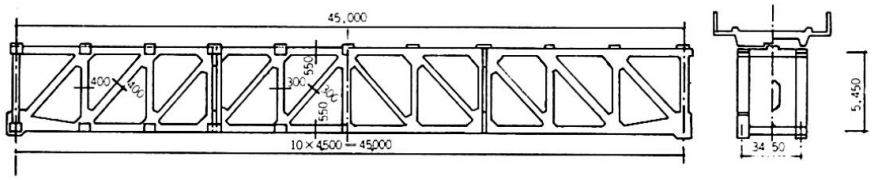

Fig. 3 45 Meter Span Plane Truss

The design compressive strength of the concrete used in the precast members was 800 kg/cm^2 and this was attained through autoclave curing. The design strength of the in-situ concrete for joining members at the panel points was 600 kg/cm^2 and it was possible to obtain this strength by using a high performance water reducing agent and without special curing.

Fig. 4 shows the precast members in a 45 meter span. Each of the plane trusses is made up of 3 upper and 3 lower chord members with the diagonal and

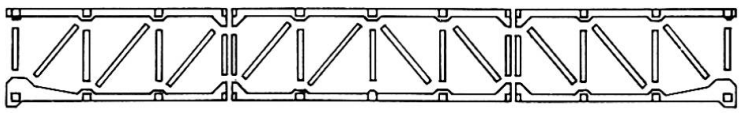

Fig. 4 Precast Truss Members

vertical members all being separate components. With 58 members in each 45 meter span, a total of 384 precast members were manufactured. The largest of these were the upper and lower chord members in the central blocks which were 18 meters long and weighed 15 tons.

5. Manufacture of Precast Members

5-1 Mix Proportions of Concrete

In selection of the aggregates, fine and coarse aggregates with a high silica content were chosen because silica forms high strength crystals during autoclave curing.

The concrete mix used a 0.30 water-cement ratio with a cement content of 530 kg/m^3 and a high performance water reducing agent. While concrete using this water reducing agent has a high viscosity and is difficult to handle, the object was to obtain concrete with a slump of 12 cm $\pm$ 2.5 cm which is relatively workable and stable.

5-2 Manufacture of Precast Members

The precast members were cast in steel molds into which pre-assembled wire reinforcement cages had been arranged, great care being taken that all the splices, sheaths and steel reinforcements would retain their respective positions and directions during and after casting.

The concrete was mixed in a one cubic meter forced-type mixer and placed in the molds using a screw bucket. It was compacted using a relatively large amplitude vibrator inserted into the concrete at close intervals and then the surface was smoothed off immediately.

After steam curing and after confirming that the compressive strength was over 300 kg/cm^2, the members were stripped from the molds and the surface chipped to give a rough finish for later jointing. A temporary prestress of 30 kg/cm^2 was introduced to prevent crack formation during handling, long distance transportation and autoclave curing. The truss members were then cured in an autoclave as shown in Photo 2.

5-3 Autoclave Curing

After concreting, the members were first steam cured at a maximum temperature of 65°C for about 12 hours to shorten the time before stripping and to increase the effectiveness of the subsequent autoclave curing.

The members were cured in an autoclave for 20 hours at a temperature of 180°C and a pressure of 10 atmospheres as shown in Fig. 5. Heating to this temperature was carried out gradually at the rate of 10°C/hour over 15 hours and after curing, the members were allowed to cool gradually at a rate of 4°C/hour over 20 hours to prevent crack formation. The autoclave was 31 meters long and had a diameter of 3 meters.

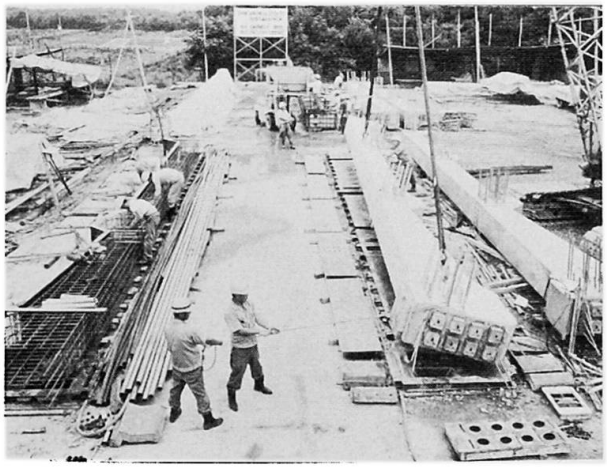

Photo 1 Manufacture of Members

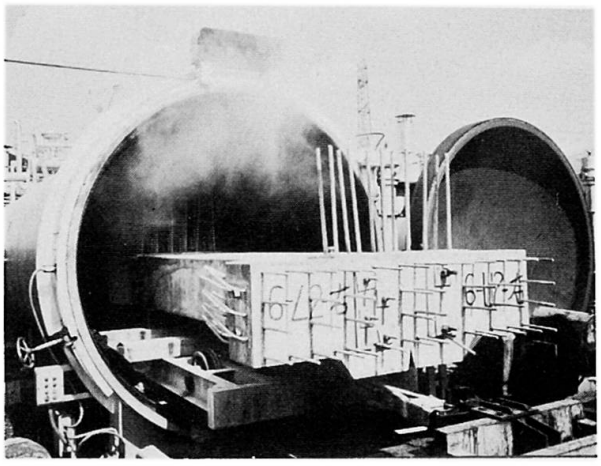

Photo 2 Autoclaving of Members

5-4 Compressive Strength of Concrete

The design compressive strength of the concrete was 800 kg/cm^2, but as a result of adding an additional factor of 1.1, the average strength attained after autoclave curing was 960 kg/cm^2, with a variation factor of 4.6%

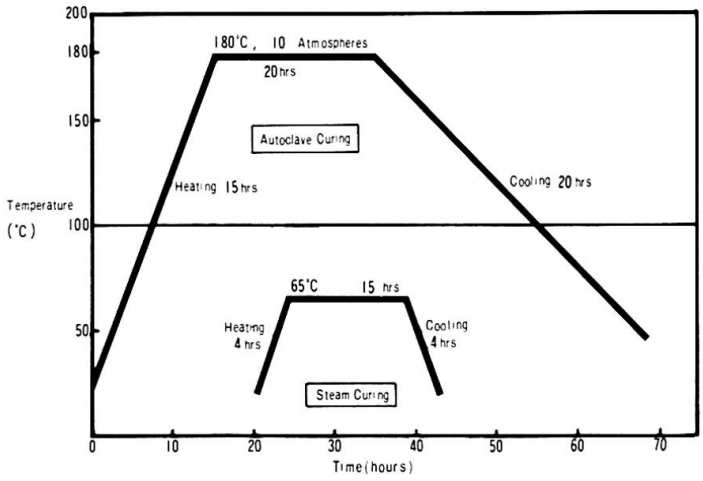

Fig 5. Autoclave Curing Cycle

6. Erection

Various methods were considered for the erection of the trusses, but in view of the site conditions and safety factors involved in the erection of such heavy concrete members, the 'all staging-pull out' method was finally adopted.

6-1 Assembly of Members

The precast members were carefully assembled into plane truss blocks on the ground and joined together at the panel points with in-situ 600 kg/cm^2 concrete and then prestress was introduced in the verticals. Measurement after assembly showed an accuracy of within $\pm$ 7 mm.

6-2 Preliminary Assembly and Joining of Space Trusses

The assembled plane trusses were raised to the verticle and then lifted up to the erection staging by crane where they were assembled into space truss blocks, being temporarily joined by lateral steel bracings. These space truss blocks were then individually pulled into position along the erection girders.

After arranging the splice reinforcements and inserting the prestressing tendons into the sheaths, the space truss blocks were joined together, three as a time, using 600 kg/cm^2 in-situ concrete and then tension was applied from one direction only, first in the upper chords and then the lower chords, to form a 45 meter span.

The space trusses were positioned with zero camber because there was a calculated camber of 12.8 mm at full dead-load after creep deformation.

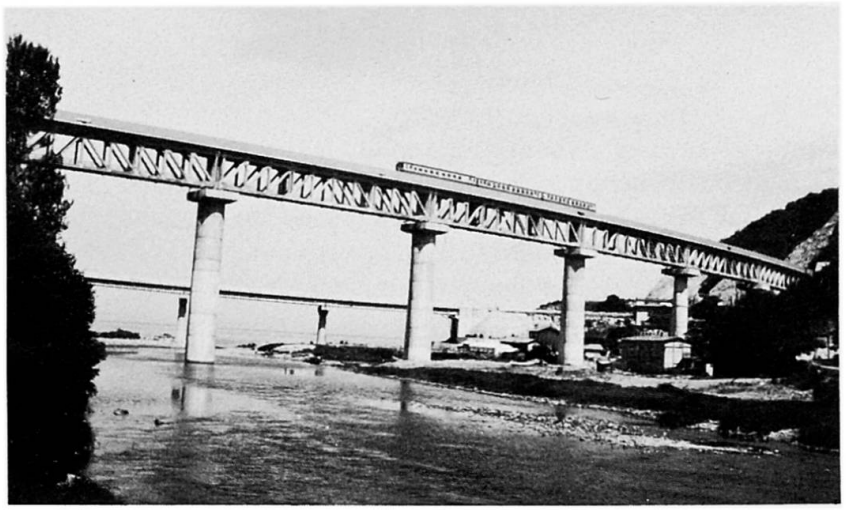

Photo 3 Completed PC Truss Railway Bridge

6-3 Earthquake Proof Walls and Deck Slabs

After the trusses had been secured to the shoes, lateral beams and earthquake proof walls were cast in position and then joined to the trusses using in-situ concrete.

The slabs were precast blocks, 4.5 meters long and 5.8 meters wide and were effectively made into a single continuous deck by the introduction of approximately 10 kg/cm^2 prestress in the longitudinal direction.

SUMMARY

The Akkagawa railway bridge, overall length 305 meters and individual span length 45 meters, is a precast, prestressed concrete bridge in which the Howe truss and extremely high strength concrete have been used. The precast members of the trusses were cured in an autoclave and attained a concrete strength of over 900 kg/cm^2. They were then assembled into space trusses using in-situ concrete and then post-tensioned.

RESUME

Le pont ferroviaire d'Akkagawa, d'une longueur hors-tout de 305 mètres et d'une longueur de travée individuelle de 45 mètres, est un pont en béton précontraint et préfabriqué où des treillis du type Howe et un béton extrêmement résistant ont été utilisés. Les éléments préfabriqués des treillis furent traités dans un four autoclave, et atteignirent une résistance de béton de plus de 900 kg/cm^2. Ils furent ensuite assemblés en treillis en utilisant du béton coulé sur place, puis mis sous précontrainte.

ZUSAMMENFASSUNG

Die 305 m lange Akkagawa Eisenbahnbrücke, eine Fachwerkbrücke nach dem Howe'schen System, besteht aus vorfabrizierten vorgespannten Bauteilen aus höchstfestem Beton. Die Einzelspannweiten betragen 45 m. Die Bauteile wurden nach Betonieren im Autoklav gehärtet, sodass ihre Betonfestigkeit 900 kg/cm^2 überstieg. Diese vorfabrizierten Bauteile wurden auf der Baustelle unter Verwendung von Ortbeton zusammengebaut und vorgespannt.

Manufacture and Erection of Prestressed Concrete Truss Bridge

Exécution et montage d'un pont à treillis en béton précontraint

Herstellung und Montage einer vorgespannten Spannbeton-Fachwerk-Brücke

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It is an increasing tendency that P. C. bridge spans will be made longer and longer. The conventional P. C. bridge structure has an economical disadvantage, for it must support itself its heavier weight with the span elongated. Adoption of the truss structure, therefore, has been well considered. Precast-concrete is more desired than concrete usually casted in place.

JNR successfully erected in 1973 the Iwahana Bridge on the New Sanyo Line, the first P. C. truss bridge in Japan (See Photo 1). In building the bridge, precast-concrete members produced in the factory were carefully assembled with adhesive agent on the staging (See Photo 2).

One of the most remarkable features is that truss bridges can be erected safely and without difficulty even over the deep valley or during the rainy season. With the completion of the Iwahana Bridge, JNR started studying on how to use the cantilever erection method in the construction of P. C. truss bridges -- a next step to the staging erection method.

The cantilever erection method brings about the following problems:

1. Members should be firmly fixed, the joint gap between members and their directions should be accurately adjusted, when temporarily overhanged.

Photo 1. The Iwahana Bridge, the first P. C. truss bridge in Japan

2. Mortar of high quality should be developed -- the mortar of high strength at early ages enough to put pre-stress into members at initial stages.

In order to solve these problems, JNR has developed a new assembling method, the so-called "AB-Plate Method." (See Fig. 1)

Used under this method are A-plate and B-plate. Joint gap between members and their directions are adjusted with the A-plate attached directly to the members. The B-plate is a splice plate, fixed through the A-plate.

By this method members can be temporarily assembled prior to the erection of the P. C. truss bridge, as in the case of the steel truss bridge. Once adjusted in position with the A-plate and B-plate, there is hardly required of each member to be re-adjusted.

Photo 2. The Iwahana Bridge, erected on the staging

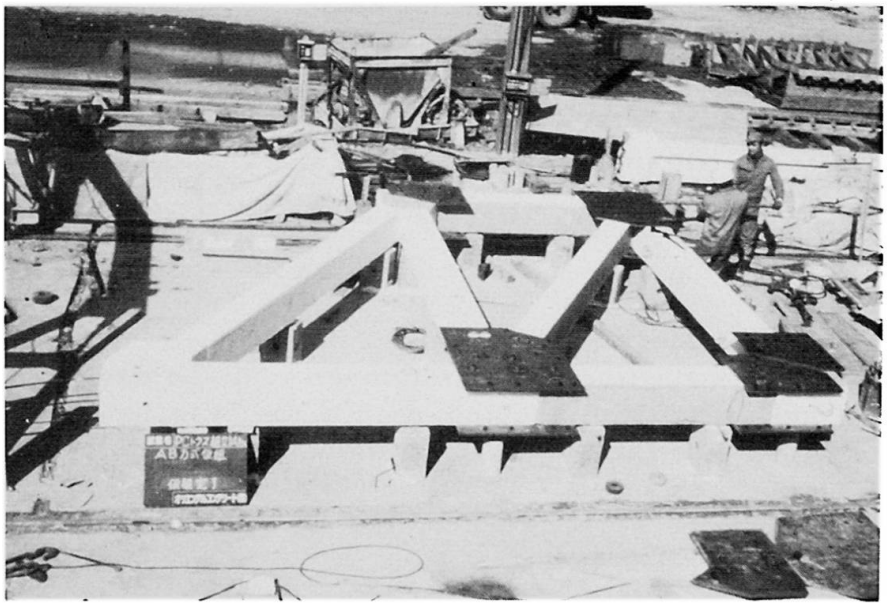
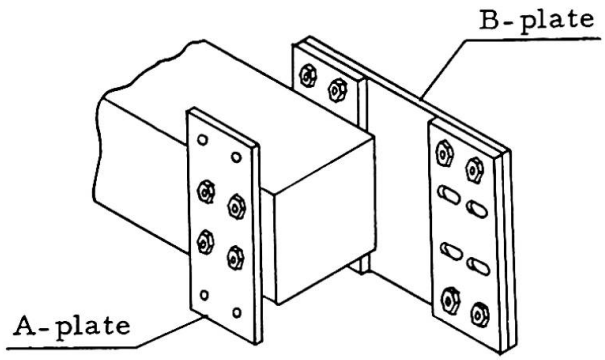
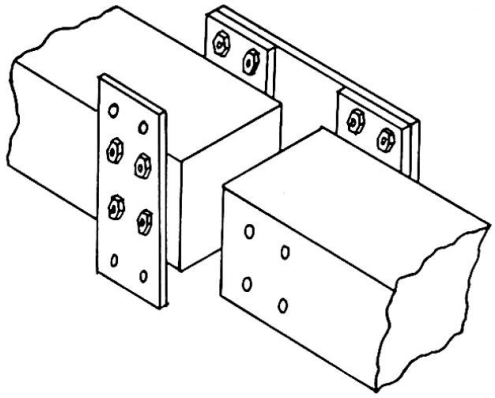


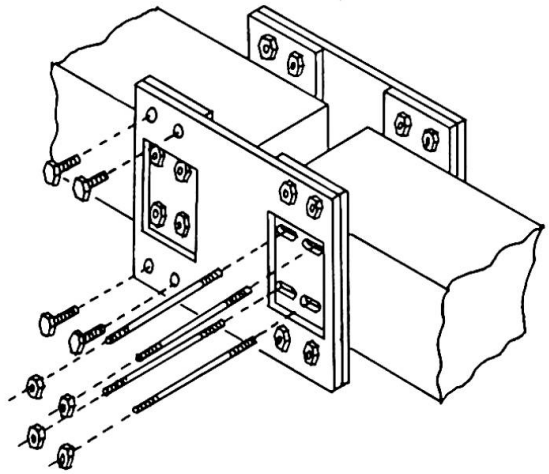
Photo 3. Truss Panel after Temporary Assembly



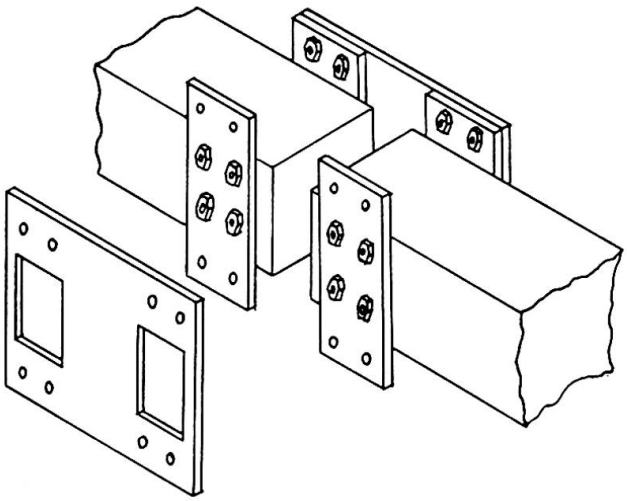
1. The one A-plate and the other A-plate joined to the one side B-plate beforehand are attached to one member.



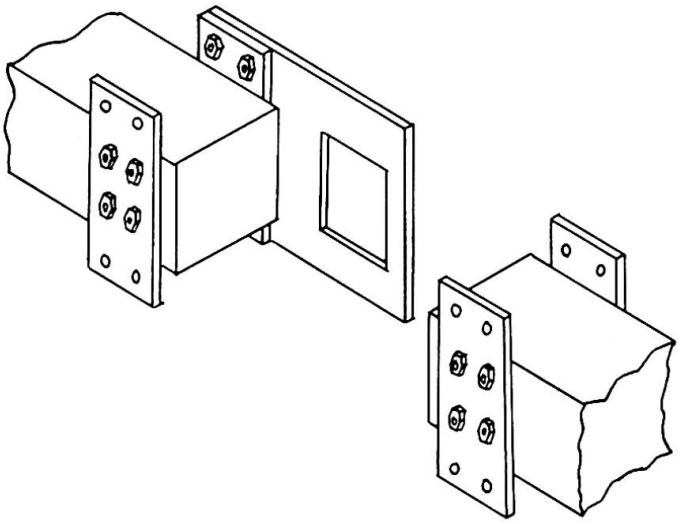
2. The joint gap between two members and directions of them are adjusted.



3. The remaining B-plate and A-plate are attached to the other member.



4. After the relative positions of both members are adjusted in this way, the B-plate on one side is detached.



5. The bolts which join the B-plate to the A-plate are taken off on one side. At this time, all the A-plates and one B-plate should not be detached. All the truss members are taken to pieces in this way, carried to the job site and then set up.

Fig. 1 Temporary Assembly Order of the AB-Plate Method

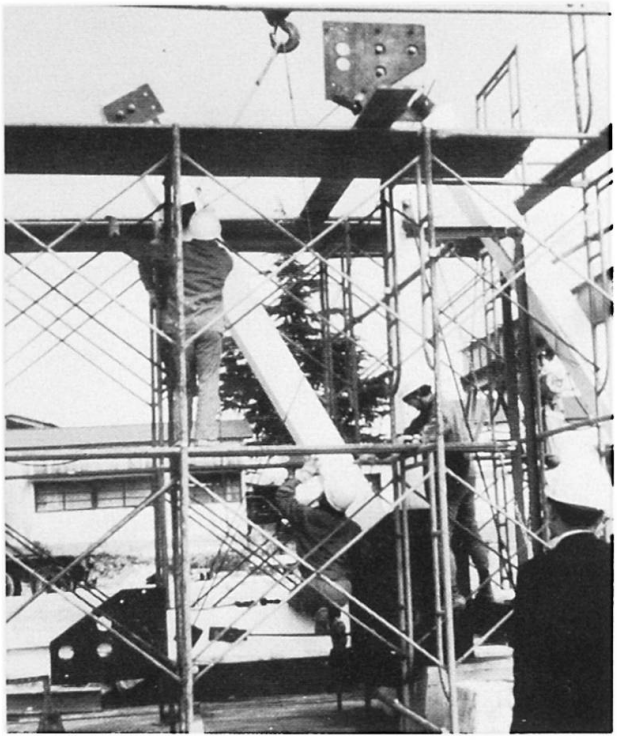


Photo 4. Assembling Test

Tests on assembling error and time were conducted by this method with the Howe type truss, 2.5 meters long and 4.25 meters high (See Photos 3 and 4). The result is as follows:

Assembling Error

This is given as the difference between 1) and 2), as shown in Table 1 -- 8 mm of the upper panel, almost 0 mm of the lower panel, and 8 mm of the truss height.

Assembling Time

This test was repeated twice, the first taking 137 minutes and the second, 105, with the time lost for adjusting the length of wire rope, as shown in Table 2. Experiences of work would reduce the 105 - 137 minutes to 90 - 105, and 30 - 35 minutes to assemble one triangle of panel.

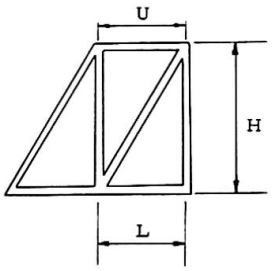
The good results the AB-Plate Method produced both in the assembling error and time will make assured that it is not necessary for all the truss panels to be assembled temporarily in advance, but only 2 or 3 at a time. The position of members displaced during the course of erecting the bridge can also be corrected with ease and whenever necessary.

JNR is making strenuous efforts to improve this method and has a plan for putting it to practical use in the construction of the Yakkan-gawa Bridge on the Nippo Line in Kyushu.

Table 1 Assembling Error

(in mm)

	U		L		H	
	Measured Value	Δ	Measured Value	Δ	Measured Value	Δ
Designed Length	2 500		2 500		4 250	
1) After Temporary Assembly	2 490	- 10	2 495	- 5	4 252	2
2) After Practical Erection	2 498	- 2	2 495	- 5	4 260	10



U : Upper Panel Length

L : Lower Panel Length

H : Height

Δ : Remainder

Table 2 Assembling Time

(in minutes)

Order	Members to be erected	1st time	2nd time	Remarks
1	Lower chord member	30	30	Time for setting up the stay is included.
2	Diagonal member	55	10	At the 1st time, it took a long time for adjusting the length of the hanging wire rope. At the 2nd time, the lever block was attached to the wire rope, so, it took shorter time.
3	Lower panel point	10	13	
4	Vertical member	10	11	
5	Upper panel point	17	13	
6	Upper chord member	15	28	At the 2nd time, the space where the upper chord member should be inserted was too short. As a result it took long time to adjust the joint gap.
Total		137	105	

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2. MACHIDA, F., et al. : Design and Execution of Iwahana P. C. Truss Bridge (1), Proc. PCEA, Vol. 17, No. 3, June 1975, pp. 40 - 55 (in Japanese).

3. MACHIDA, F., et al. : Design and Execution of Iwahana P. C. Truss Bridge (2), Proc. PCEA, Vol. 17, No. 4, Aug. 1975, pp. 4 - 14 (in Japanese).

SUMMARY

JNR successfully erected the Iwahana Bridge in 1973 by the method of staging, the first through type prestressed concrete truss bridge in Japan. It is more advantageous in many points for a prestressed concrete truss bridge to be erected by the cantilever method with precast concrete members assembled, and the wider use of this method is expected. In order to assemble precast concrete members more efficiently and with less trouble, a new method has been developed - the so-called "AB-plate-method".

RESUME

Les Chemins de Fer Nationaux du Japon ont construit avec succès, en 1973, le pont de Iwahana, premier pont à treillis en béton précontraint au Japon, par la méthode sur échafaudage. Il est plus avantageux en bien des cas de construire un pont à treillis en béton précontraint par la méthode du porte-à-faux avec des éléments de béton préfabriqués; on espère en élargir le champ d'application. Une nouvelle méthode appelée "méthode AB-plate" a été développée pour assembler les éléments de béton préfabriqués plus efficacement et avec moins de problèmes.

ZUSAMMENFASSUNG

Die Japanische Staatsbahn erbaute im Jahre 1973 die Iwahana Brücke, die erste durchlaufende Spannbeton-Fachwerk-Brücke in Japan, mit Hilfe eines Montagegerüstes. Unserer Meinung nach ist der Freivorbau unter Verwendung von vorgefertigten Betonelementen in vielen Fällen vorteilhafter. Aus diesem Grund entwickelten wir eine neue Baumethode, die sogenannte "AB-plate"-Methode, um mit vorgefertigten Betonelementen wirkungsvoller zu bauen.

Un exemple de pont à haubans multiples répartis avec préfabrication partielle du tablier: le pont de Brotonne

Die Brücke von Brotonne, ein Beispiel für Schrägseilbrücken mit verteilten Seilen und teilweise vorgefertigter Fahrbahn

The Brotonne Bridge, an Example of Cable Stayed Bridge with Multiple Cables and a partly Precast Deck

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Pour des portées supérieures à 200 m., les ponts en béton précontraint, construits par encorbellement, peuvent être remplacés avantageusement par des ponts à haubans multiples répartis. Ces derniers peuvent d'ailleurs être considérés comme des ponts en encorbellement à précontrainte extérieure, le bras de levier de la précontrainte étant augmenté considérablement en écartant du tablier les câbles, qui se comportent alors comme des haubans et s'appuient sur un mât qui assure leur déviation (Fig. 1).

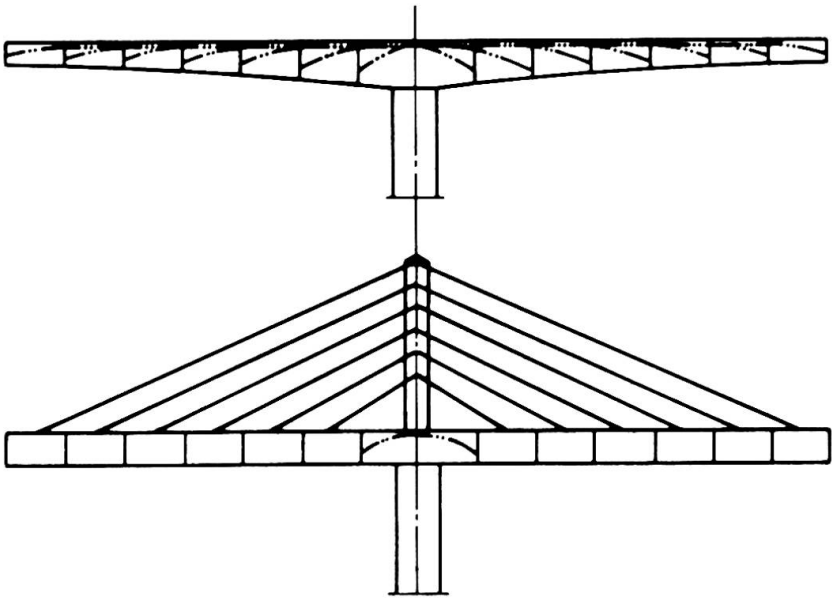


Fig. 1

Les ponts à haubans multiples répartis présentent plusieurs avantages:

- facilité de montage du tablier puisqu'on peut avancer la construction en porte-à-faux jusqu'au hauban suivant,
- simplification de la transmission des efforts, d'une part à l'ossature du tablier, d'autre part aux pylones par réduction des forces concentrées au droit des ancrages et diminution de la flexion entre points de suspension,
- commodité de remplacement des haubans en cas de détérioration sans interrompre l'exploitation de l'ouvrage,
- meilleure stabilité aérodynamique par suite de l'augmentation de l'amortissement propre du système dû au nombre élevé de haubans de longueur variable et donc de fréquences différentes.

Les haubans peuvent être constitués de câbles clos ou de câbles de précontrainte du type à fils parallèles ou toronnés.

Une suspension du type rigide, c'est-à-dire composée de câbles de précontrainte sous enveloppe métallique, injectés à la résine ou au coulis de ciment, offre une solution intéressante aux problèmes posés à la fois par la résistance à la fatigue et par la protection contre la corrosion des haubans.

En effet, à poids d'acier égal, la variation de tension sous charges alternées est réduite dans des proportions sensibles du fait de la participation de l'enveloppe métallique à la section résistante. De plus, la nature des aciers de précontrainte, dont la résistance à rupture est meilleure que celle des câbles clos, accroît encore la plage d'utilisation des tensions dans les haubans.

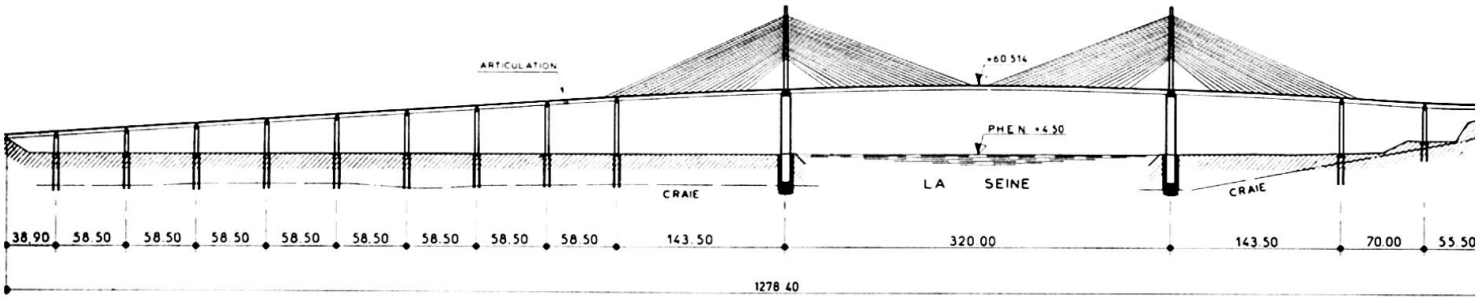


Fig. 2

La construction des ponts à haubans multiples répartis peut faire appel, de façon partielle ou totale, à la préfabrication.

Le pont de Brotonne, qui franchit la Seine à l'ouest de Paris, fournit un exemple intéressant d'un pont à haubans construit par encorbellement, pour lequel on a eu recours à la préfabrication de certains éléments du tablier. Cet ouvrage, dont les haubans sont disposés en éventail dans le plan médian de la structure,comporte une travée centrale de 320 m. (Fig. 2).

La suspension est assurée au moyen de 21 haubans constitués de 39 à 61 torons T15 enfilés dans des tubes métalliques et injectés au mortier (Fig.3).

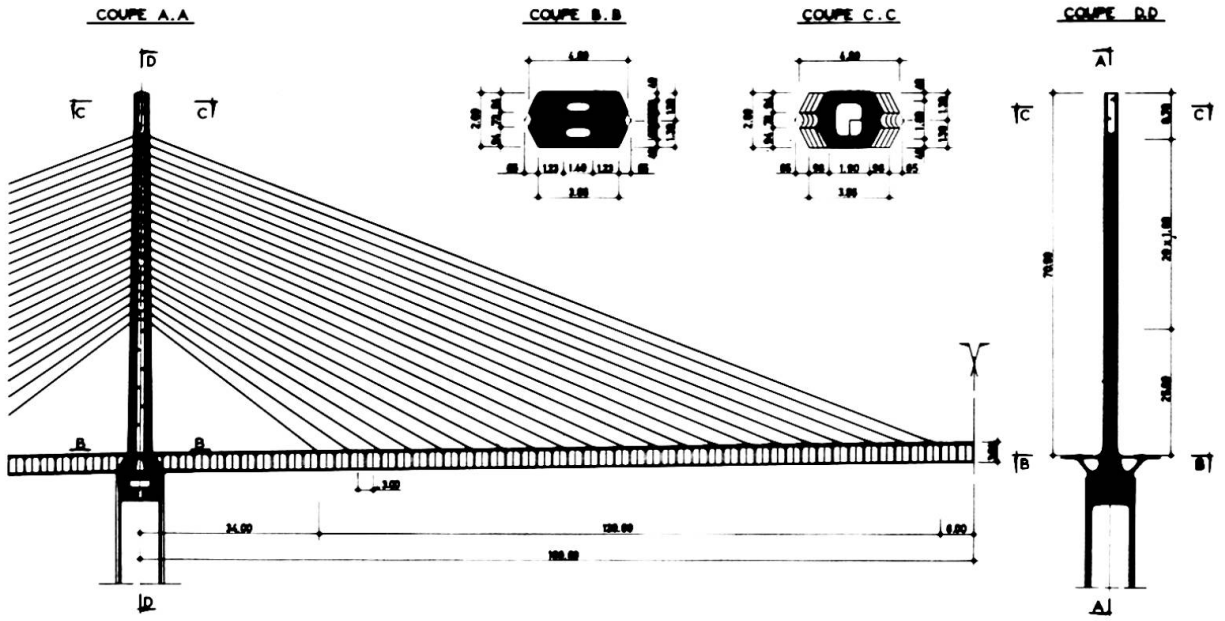
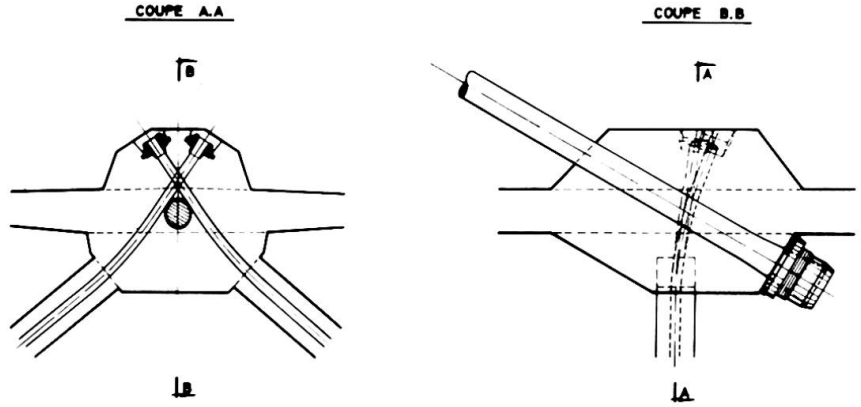
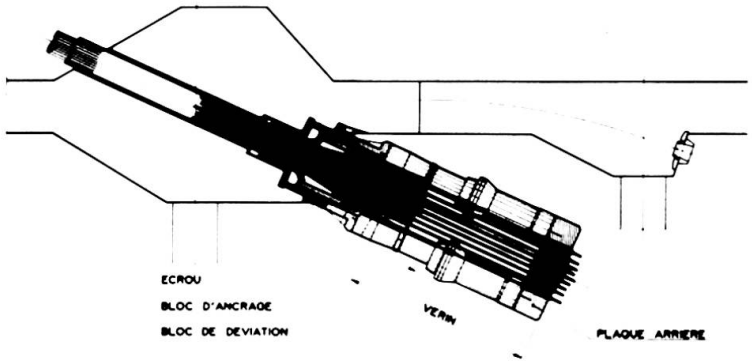


Fig. 3

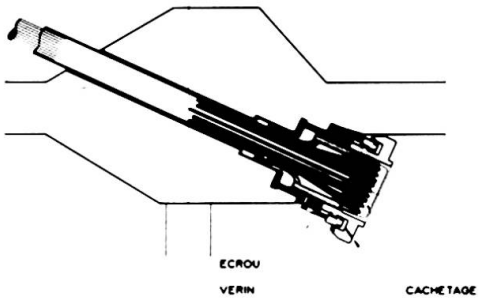
Ces haubans, qui s'appuient au droit des mâts sur des selles formant points fixes, peuvent être retendus ou remplacés à tout instant de la vie de l'ouvrage. (Fig. 4)



Ancrage des haubans



Dispositif de tension



Dispositif de retension

Fig. 4

Les mâts sont encastrés dans le tablier, lequel repose sur les pylones par l'intermédiaire d'appuis en néoprène (Fig. 5).

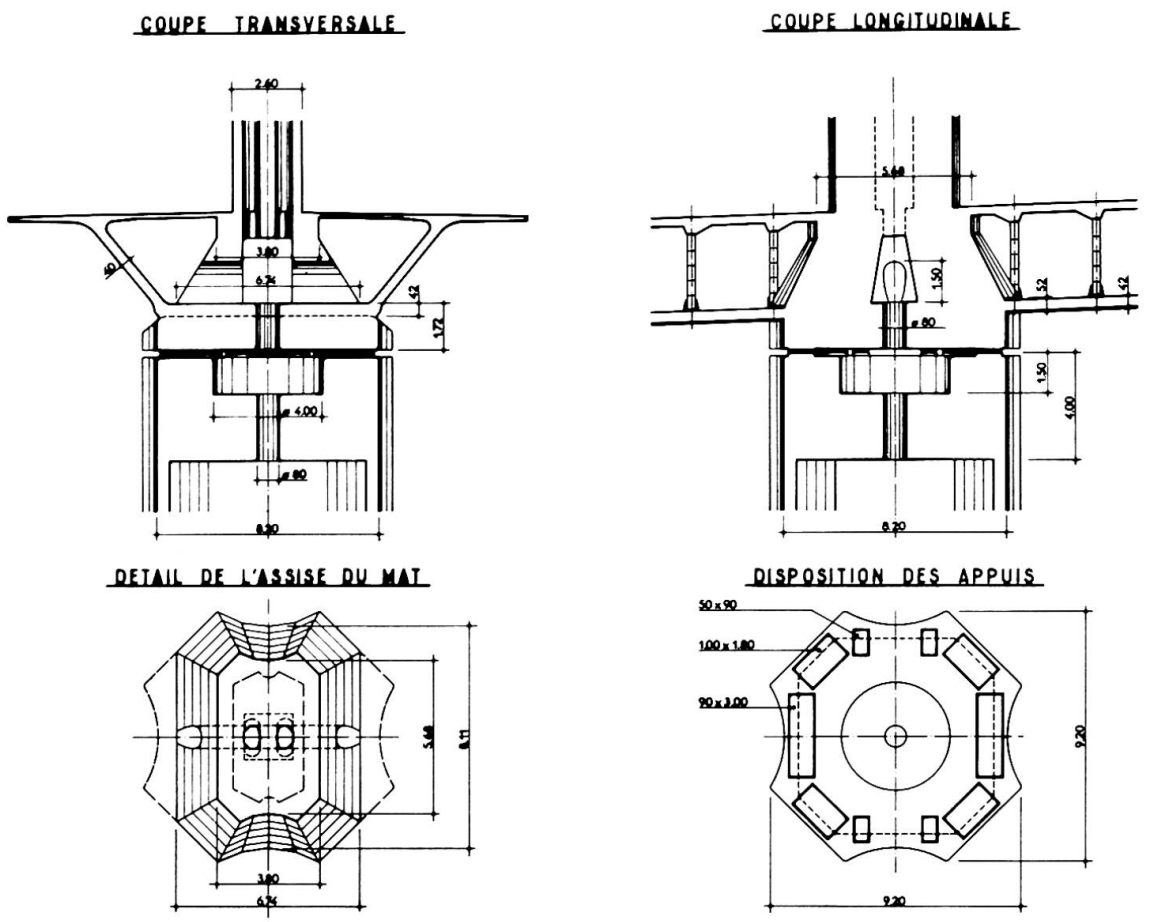


Fig. 5

La section transversale du tablier se compose d'une poutre-caisson monocellulaire raidie intérieurement par des butons obliques (Fig. 6).

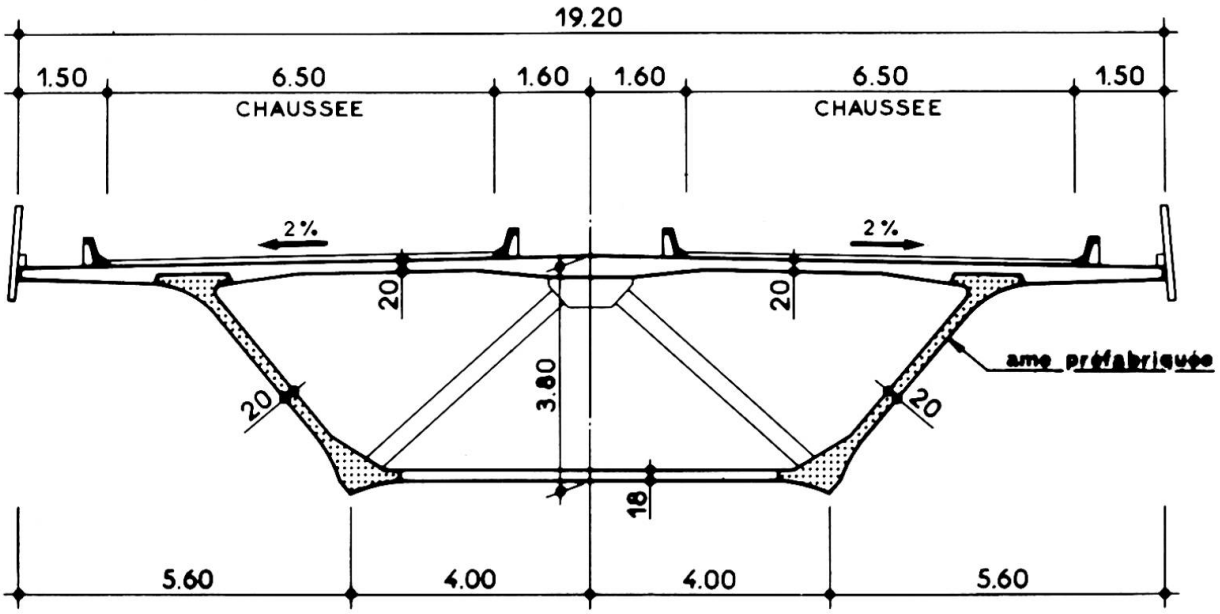


Fig. 6

Les âmes, de faible épaisseur et fortement inclinées, sont préfabriquées par paires sur un banc et précontraintes par fils adhérents (Fig. 7).

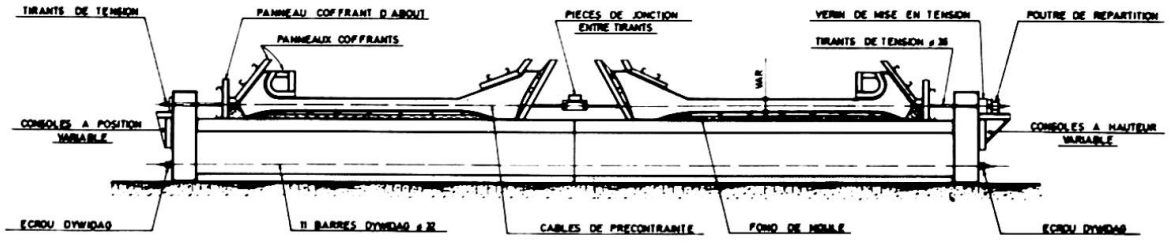


Fig. 7

Le tablier est construit par encorbellement à partir des pylones en voussoirs de 3,00 m. de longueur. Chaque voussoir est réalisé en deux étapes.

Dans une première étape, les âmes préfabriquées sont posées sur des berceaux à l'intérieur d'équipages mobiles constitués d'une charpente métallique fixée à l'extrèmité du tablier en cours de construction (Fig. 8). Les équipages sont alors règlés de façon à épouser le profil en long de l'ouvrage corrigé des déformations prévisibles.

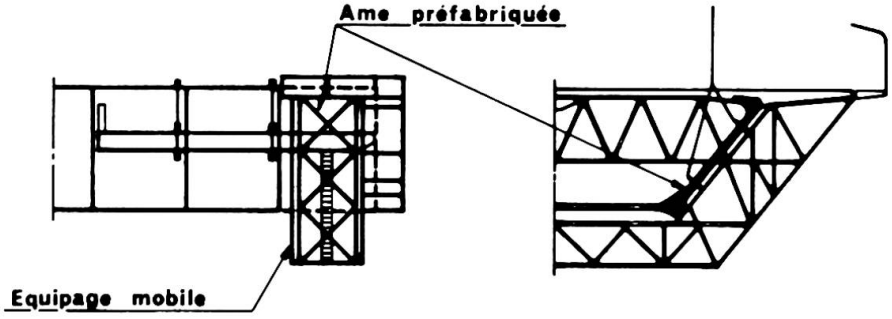


Fig. 8

Dans une seconde étape, les âmes sont solidarisées à la partie du tablier déjà construite par l'intermédiaire d'un joint maté et de barres de précontrainte provisoire (Fig. 9). Le reste de la section transversale est ensuite bétonné à l'intérieur des équipages mobiles après que ces derniers aient été suspendus aux âmes.

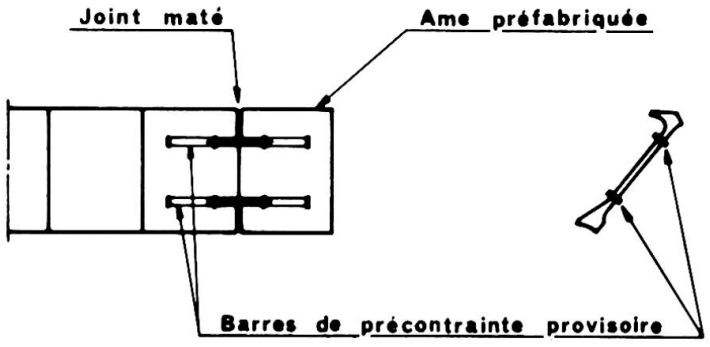
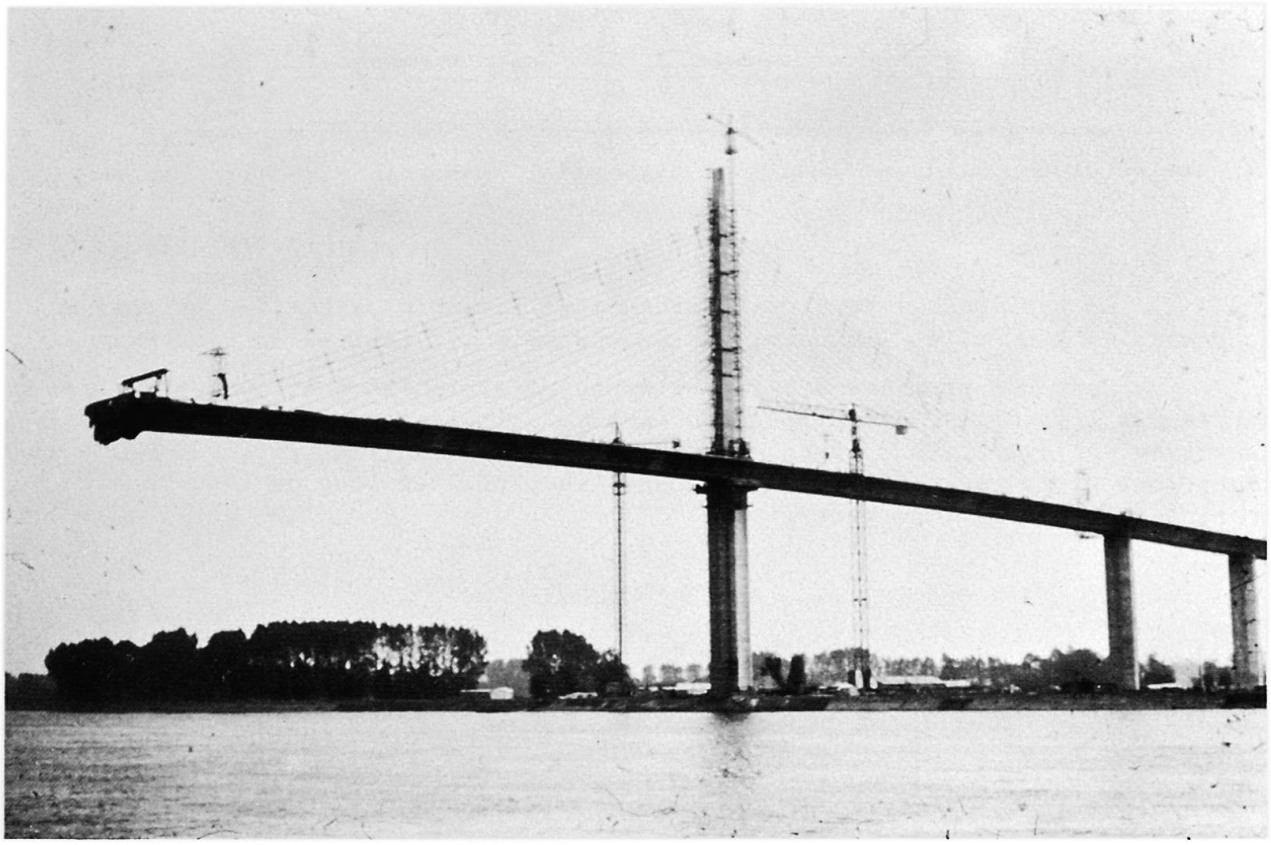


Fig. 9

Ce procédé, faisant appel à une préfabrication partielle des voussoirs au moyen d'éléments légers (les âmes pèsent 12 tonnes) a ainsi permis de simplifier les équipages mobiles de bétonnage dont le poids reste limité (35 tonnes) et de réduire le cycle de construction du tablier en permettant la réalisation de 4 voussoirs par semaine, soit 12,00 m. de tablier.



Ouvrage en construction - Fig. 10

RESUME

Le pont de Brotonne est un pont à haubans multiples répartis en béton précontraint construit par encorbellement avec préfabrication partielle du tablier, comportant une portée centrale de 320 m, qui constitue le record parmi les ouvrages de ce type.

ZUSAMMENFASSUNG

Die Brücke von Brotonne wurde im Freivorbau als Schrägseilbrücke aus Spannbeton mit verteilten Seilen und teilweise vorgefertigter Fahrbahn erstellt. Mit einer Mittenspannweite von 320 m stellt sie einen Rekord unter den Schrägseilbrücken auf.

SUMMARY

The Brotonne bridge is a prestressed concrete cable-stayed structure built in cantilever with fanned multiple cables and a partly precast deck; its 320 m long center span makes it a record of its type.

Industrielle Herstellung von Viadukten und Brücken

The Industrial Fabrication of Viaducts and Bridges

Construction industrielle de ponts et de viaducs

VUKASIN ACANSKI
GIP GRADIS LJUBLJANA
Maribor, Jugoslawien

1. Einführung

Die rasche wirtschaftliche Entwicklung, der damit verbundene Aufschwung des Lebensstandardes und die rasante Zunahme des Tourismus, haben in Jugoslawien zu einer enormen Ausdehnung des Strassennetzes geführt. Die Integration des jugoslawischen Strassennetzes in das europäische ist eine Notwendigkeit geworden.

Die jugoslawische Topographie ist sehr bewegt und bedingt daher eine grosse Anzahl von Brücken und Viadukten, das heisst, von Objekten, welche durch die rasch anwachsende Motorisierung und den damit verbundenen Ansprüchen an die Fahrsicherheit und den Fahrkomfort immer zahlreicher geworden sind. Diese Entwicklung fordert neue Baumethoden, welche technologisch einfacher und zeitsparender sind. Die kurzen Bautermine der immer zahlreicheren Kunstbauten, verlangen besser ausgebildete Fachleute und eine entsprechende technische Ausrüstung.

Die fehlenden Arbeitskräfte, sowie die bescheidene technische Ausrüstung der jugoslawischen Unternehmungen führen zur Entwicklung industrieller Bauweisen für Kunstbauten.

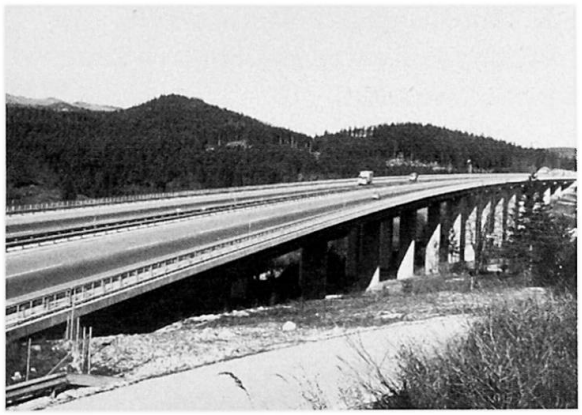


Bild 1:
Viadukt Ravbarkomanda
an der Autostrasse
Vrhnika-Postojna

Mit dem industriellen Bau von Viadukten und Brücken "System Gradis" werden folgende Vorteile erreicht:

- Verkürzung der Bauzeit
- Verminderung der Anzahl der Arbeitsstunden
- Die Fachkräfte bleiben in den Betrieben konzentriert wodurch geordnete Lebensverhältnisse und eine Verbesserung des Lebensstandardes jedes Arbeiters entstehen

- die Arbeitsleistung wird durch die besseren Organisationsmöglichkeiten und besseren Bedingungen am Arbeitzplatz grösser
- die Eingliederung der nichtqualifizierten Arbeitskräfte in den Arbeitsprozess wird einfacher,
- Verminderung der Regiestunden am Arbeitsplatz
- Kleinere Spesen für Feldarbeit und getrenntes Leben
- die Einrichtung der Baustelle geht schneller und einfacher, dadurch kleinere
- finanzielle ausgaben
- die Hilfsobjekte, w.z.B. die Lager, Betonaufbereitungsanlagen, Arbeitslogis etc. sind weniger aufwendig und zum Teil Überflüssig
- da die mechanische Ausrüstung grösstenteils in den Werkstätten konzentriert ist, kann sie besser genützt werden. Dei Transporte, die Montage- und Demontage dieser Geräte, sind nicht mehr notwendig,
- die Qualität des Bauens wird die bessere Kontrolle erhöht,
- die Wettereinflüsse auf den Bauvorgang sind unbedeutend,
- der Brückenbau ist von Arbeiten am Strassentrasse nicht mehr abhängig.

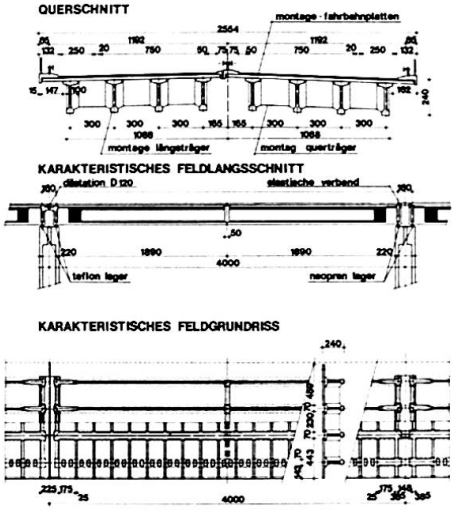


Bild 2: Typisches Brücken und Viaduktenfeld zusammengestellt mit industriel hergestellte Montage Elemente

Dies sind nur ein Teil der Vorteile dieser Bauweise. Diese Technologie des Brückenbaues hat jedoch auch ihre eigenen spezifischen Bedingungen:

- die entsprechende Ausrüstung für die Montage der vorfabrizierten Elemente
- die Projektierungsarbeit ist aufwendiger und anspruchsvoller,
- die Technologie bedarf einer präzisen Planung der einzelnen Bauphase
- die ökonomischen Vorteile sind von der Anzahl der typischen Objekte und Kontinuität des Bauvorganges abhängig.

Der Zweck dieses Artikels ist die Erläuterung der Methode für die Erstellung von Viadukten und Brücken auf der Autostrasse Šentilj-Razdrto, welche nach dem System "Gradis" aufgebaut wurden.

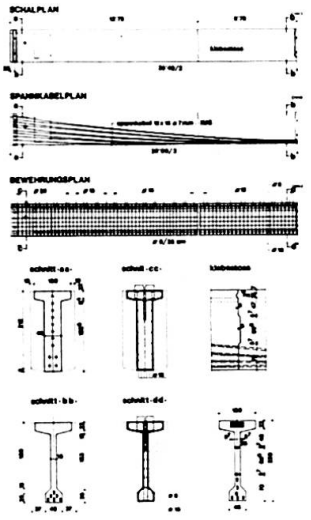


Bild 3: Industriell erstellte Montage-Träger für Brücken und Viadukt.

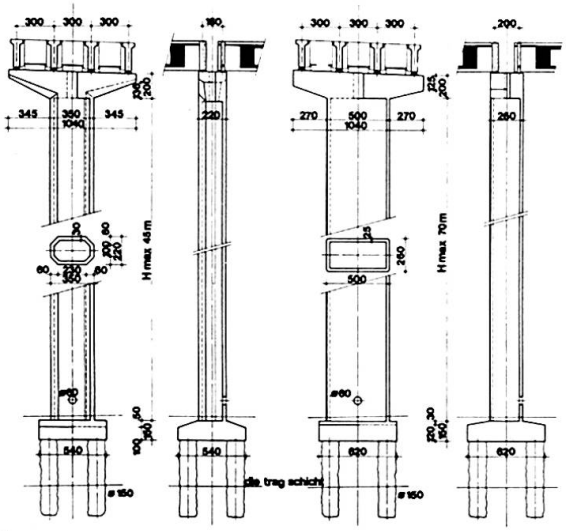


Bild 4: Typisches Stützen formen für Viadukte

2. Erläuterung der Konstruktion eines typisierten Objektes

Der Entwurf dieser statischen Konstruktion wurde auf Grund ökonomischer und technischer Analysen erstellt, welche durch folgende Studien entstanden sind:

Bild 5: Trägerdepo im Betonwerk

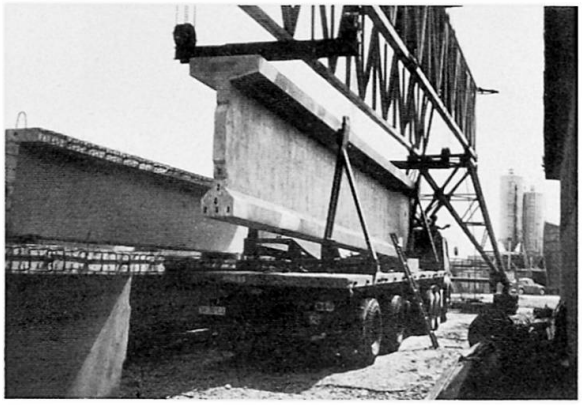


Bild 6: Trägeraufladung auf Transportmittel

- Bestimmung der optimalen Spannweite zwischen den Pfeilern bei einer durchschnittlichen Höhe von H = 35 m
- Auswahl des statisch günstigsten Querschnittes des Oberbaues,
- Bestimmung des technologisch günstigsten Vorganges für die Montage der Brückenelemente in Bezug auf das Gewicht und die Abmessungen
- die technische und ökonomische Ausführung der Bauelemente,
- die Transporte der Bauteile innerhalb und ausserhalb der Baustelle.

Die Resultate dieser Studien zeigten, dass in Anbetracht der gegenwärtigen und zukünftigen Mobilität, sowie der ökonomischen und spezifisch topographischen Verhältnisse in Slowenien, diese Konstruktion, welche aus frei aufliegenden Feldern mit einer Spannweite von L = 40 m bilden, die günstigste typisierte Brückenbaukonstruktion ist.

Eine bestimmte Zahl der frei aufliegenden Felder wird durch Gelenkverbindungen zusammen verbunden. Dadurch formen wir sogenannte "Bremseinheiten", welche durch Dilatationsfugen getrennt sind. Die Länge einer Einheit ist von der Höhe der Stützen abhängig und beträgt im max. 400 m.

Das Tragssystem des frei aufliegenden Feldes besteht aus der Fahrbahnplatte, welche durch Längsträger unterstützt ist. Die Längsträger sind durch End- und Feldquerträger zusammen verbunden. Die Längsträger liegen in Abständen von 3.0 - 3.5 m auf Neoprenlagern, welche die Belastung auf die Pfeiler übertragen.

Die Pfeiler sind elastisch in die Fundamente eingespannt. Die Fundamentkonstruktion wird, je nach Baugrundverhältnissen, flach gewählt oder mit

der Grundpfählung kombiniert.
Der Querschnitt der Zwischenpfeiler ist konstant und verbreitert sich konsolartig unter der Oberbaukonstruktion.
Die Widerlager werden je nach topographischen bzw. geologischen Verhältnissen gewählt.

Bild 7: Transport eines Montage-Trägers auf die Baustelle

3. Technische Ausführung der Montage-Elemente

Die Träger

Die Träger, welche im Querschnitt die Form eines doppelten "T" aufweisen, sind 2.20 m hoch. Die theoretische Spannweite beträgt 38.20 m, die Gesamtlänge 39.00 m, bei einem Gewicht von 78.0 Mp. Sie werden in einer Stahlschalung betoniert. Jede Schalung ist mit Metallwänden versehen, welche den Träger in drei Elemente unterteilen.

Der Betoniervorgang alle drei Teile jedes Trägers wird gleichzeitig in kleinen Schichten vorgenommen. Der Beton wird mit Schallungsvibratoren, teilweise mit Nadelvibratoren, verdichtet.

Die Träger werden aus Beton MB 400 erstellt. Die Vorgebundenen Armierungskörbe, in welchen auch die Rippenblechröhre für die Vorspannkabel eingebunden sind, werden in die Schalung versetzt. Die Armierungskörbe werden mit speziellen Einrichtungen angefertigt. Vor Betonierbeginn werden die Kabelrohre mit "Arbeitskabel" ausgestopft, um eventuelle Deformationen der Kabelrohre zu vermeiden. Diese "Arbeitskabel" werden nach dem Abbinden des Betons herausgezogen und können wieder verwendet werden.

Die einzelnen Elemente des Trägers wiegen 26 Mp und werden nach Erreichung der Beton-Druckfestigkeit von 150 kg/cm^2 mit dem Kran deponiert.

Querträger, Fahrbahnplatten, Gehweg-Elemente

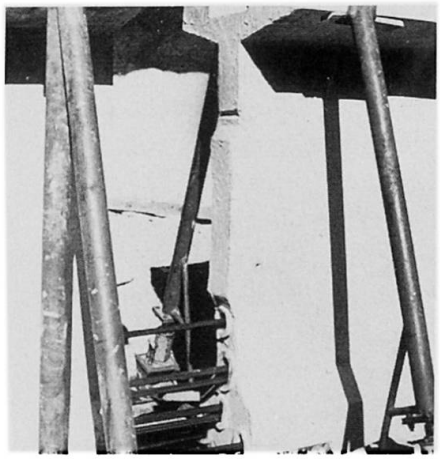


Bild 8: Detail des Kleberstosses

Bild 9: Vorspannen von IMS Vorspannkabels

Die Querträger in Rechteckform bestehen aus Montage-Elementen, welche zwischen den Längsträgern eingebaut werden. Die Fahrbahnplatten sind ebenfalls als vorfabrizierte Elemente ausgebildet. Die Stärke beträgt 20 cm, die Breite 1.75 m, die Länge ist von der Konzeption des Objekt--Querschnittes abhängig. Die Gehwegelemente sind 2.00 m lang und nach den Vorschriften für die Autostrassen Entworfen.

Alle Elemente werden in den Werkstätten in der Stahlschalung erstellt und schlaff armiert. Die Armierung wird wie bei anderen Montageelementen vorgängig in Armierungskörben zusammengebunden und in die Schalung verlegt. Die Betonqualität ist für alle Montageelemente MB 400.

4. Transport und Montage der vorfabrizierten Elemente

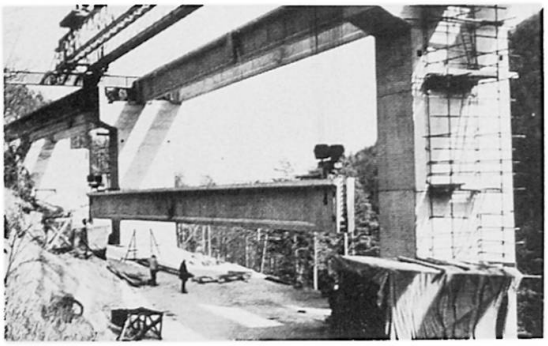


Bild 10: Konstruktion für die Montage der Träger über die Klebepiste

Der Transport am Erstellungort und der Aufland auf das Transportmittel erfolgt durch einen Portalkran.
Für den Transport der Trägerelemente werden meistens Lastwagen mit Anhänger verwendet. Auf der Baustelle werden die Trägerelemente mittels der Konstruktion für die Montage von den Transportmitteln auf die Bahn für das Kleben und Vorspannen abgeladen. Nach Einzug sämtlicher vorgesehener Vorspannkabel erfolgt der Anstrich der Stirnflächen dem Epoxi-Kleber und das Zusammenpressen durch die hydraulischen Pressen. Nach dem Vorspannen der ersten Kabelphase wird der Träger mit der Montagekonstruktion gehoben, auf das Montagefeld transportiert und auf die Neoprenlager versetzt.

Die Montagekonstruktion ist so aufgebaut, dass sie in ihren bereich jedes Montageelement haben und verlegen kan. Die Bewegungsmöglichkeit der Montagekonstruktion ist nicht von bereits montierten Elementen abhängig.
Die Querbewegung erfolgt auf den Stahlbahnen, welche auf den Pfeilern fiksirt sind.

Die Träger sind mit den Kabeln des Systemes IMS vorgespannt.
Jedes Kabel besteht aus 16 Drähten Ø 7 mm. Die Drähte sind aus hochwertigem Stahl der Qualität Sm/S02 = 170/150 kp/mm^2.

Nach der Montage der Längsträger folgt das Einfügen der Querträger welche mit der mobilen Montagekonstruktion ausgeführt wird.

Die Montage der Fahrbahnplatten erfolgt von beiden Enden der Brücke her in Richtung gegen die Mitte. Die Platten werden mit Lastwagen auf die Baustelle Transportiert. Der Autokran versetzt sie direkt an die vorgesehene Stelle.

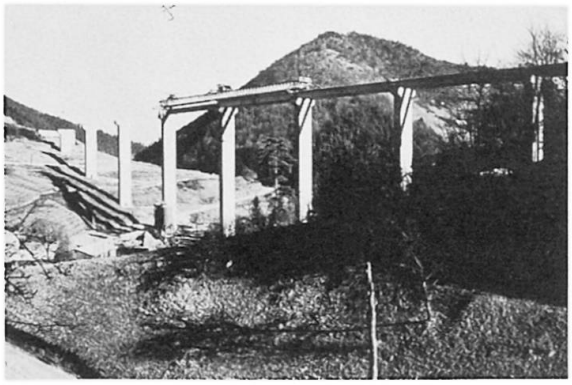


Bild 11: Ansicht auf die Montage-konstruktion und Montage-Vorgang

5. Schlusswort

Der dargestellt Vorgang der Erstellung von Viadukten und Brücken ist das Resultat mehrjärigen Entwicklung dieser Bauweise mit Montageelementen.

Die Erfahrungen, welche die Projektanten und die Ausführenden in diesem mehrjärigen Arbeitsprozess gesammelt haben, garantieren eine technisch und ökonomisch einwandfreie Ausführung der Objekte.

Es ist aber klar, dass bis heute noch keine universale Bauart erfunden wurde. Durch unsere Unternehmung "Gradis" werden immer neue Technologien studiert, welche sich ergänzen, auf spezifische Verhältnisse anpassbar sind und unserem Unternehmen die Konkurenzmöglichkeit bei jeder Art des Brückenbaues ermöglichen.

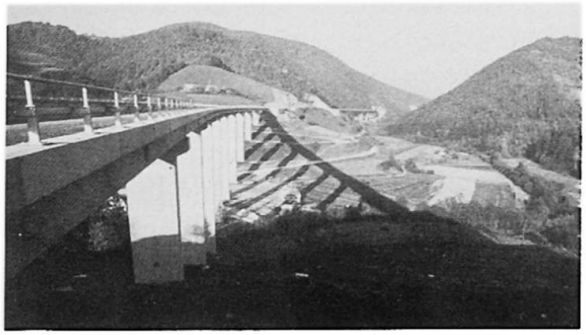


Bild 12: Viadukt Škedenj I^{I} an der Autostrasse Hoče-Levec

ZUSAMMENFASSUNG

Die Topographie in Jugoslawien verlangt die Erstellung unzähliger Brücken und Viadukte. Die industrielle Herstellung von Kunstbauten bringt grosse technische und wirtschaftliche Vorteile. Dieser Artikel befasst sich in Detail mit der Technologie der industriellen Herstellung von Viadukten und Brücken nach dem System "Gradis". Er erläutert die Konstruktion des Standard-Objekte, die Technik der Ausführung und des Transportes sowie der Montage der Bauelemente.

SUMMARY

Due to the topography in Yugoslavia, the construction of highways requires a great number of viaducts and bridges. The industrial fabrication gives many technical and economical advantages. The paper deals in detail with the technology of industrial fabrication of viaducts and bridges according to the system "Gradis": construction standard, construction techniques, problems of transport and assembling of construction elements.

RESUME

La topographie de la Yougoslavie entraine la construction de nombreux ponts et viaducs. La construction industrielle apporte de grands avantages techniques et économiques dans les travaux routiers. Ce rapport présente la technologie de la construction industrielle de ponts et de viaducs selon le système "Gradis": norme de construction, technique de construction, transport et montage des éléments de construction.

VII

Progrès dans les maisons hautes *

Progrès dans le dimensionnement des poutres à âme pleine et en caisson, en acier

Progrès dans la charge des ponts

Fortschritte bei Hochhäusern *

Fortschritte in der Dimensionierung von stählernen Vollwand- und Kastenträgern

Fortschritte bei der Erfassung der Belastung von Brücken

Progress on Tall Buildings *

Progress in the Design of plate- and box-girders in Steel

Progress on Bridge Loading

* Unfortunately the report "Progress on Tall Buildings" was not delivered by the time the whole work was given to the printer.

The Editor

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Progress in the Design of Steel Plate- and Box Girders

Progrès dans le dimensionnement des poutres à âme pleine et en caisson en acier

Fortschritte in der Dimensionierung von Vollwand- und Kastenträger in Stahl

CHARLES MASSONNET
Chairman of Working Commission II of IABSE
Professor at University of Liège
Liège, Belgium

1. INTRODUCTION.

It is not the first time that the undersigned has the duty to report about progress in the design of plate and box girders in steel. He was invited to establish the General Report in this theme at the New-York 1968 Congress of IABSE A7 and, in 1971, at the request of the late professor BEER, at that time chairman of Commission 2 (Steel) of IABSE, he organized in London, jointly with professors L.S. BEEDLE and K.C. ROCKEY, a Colloquium entitled :

Design of plate and box girders for ultimate strength

whose proceedings, published in July 1972 |C7|, were favourably received by the members of IABSE. This important source of references will be mentioned in the references as : IABSE London 1971 Colloquium.

This field of research is still in a state of rapid evolution. One main reason is the big accidents which struck four large steel box girders during their erection (|B1| to |B9|). These accidents triggered an important research effort in the countries most concerned, namely Great Britain and the Federal Republic of Germany with some additional work in Australia, Belgium, Japan and other countries. In Great Britain, a Committee, called the Merrison Committee, was appointed by the government and published its conclusions |D4| in 1972, in a form usually called the Merrison rules. The research on the strength of box girder bridges continues very actively in Great Britain, with the central activity at Imperial College, London. This Institution has organised from July 6 to 9, in consponsorship with the Institution of Structural Engineers, the Royal Institution of Naval Architects and Constrado an International Conference on Steel Plated Structures. The 22 papers cited from this source will be abbreviated in the references as "ICSPS paper N° X".

In Western Germany, a special subcommittee called "Unterausschuss "Stabilität" and especially its Working Group "Plate Buckling" was set up. The author participates, as foreign member, with professor P. DUBAS, to the work of this Committee, while a series of experimental and theoretical researches are still under way in Darmstadt, Hannover, Karlsruhe, Braunschweig, Bochum and Berlin. Finally, the author has the responsibility to lead a subcommittee of Committee 8 (Stability) of the European Convention for Constructional Steel-

work (ECCS). This Work.Group (8/3) devoted to the study of buckling of plates has, up to now, concentrated his efforts on the most urgent problems, namely the design of plate and box girders. His work up to June 1976 was summarized into two State-of-Art Reports that have been included in the

Manual on the Stability of Steel Structures

which forms the Introductory Report of the Triple

Colloquium on the Stability of Steel Structures,

which will take place in Tokyo (9.9.1976), Liège (Belgium) (13 to 15.4.1977) and Washington (17 to 19 May 1977) under the Joint Sponsorship of IABSE, ECCS, USSRC and Japan Stability Research Committee.
I shall try, in 25 minutes, to present you a broad survey of these extremely complicated problems. My written report will be much more complete that my oral presentation. In a subject as fluid as this one, you could not hope that I shall come out with clear-cut design rules. I shall be happy if I can review thoroughly enough the recent research literature in the field concerned, stress the progress already obtained, and show the trend of the current research effort.

2. CRITICAL APPRAISAL OF THE LINEAR BUCKLING THEORY OF PLATES AND THE LESSONS TO BE DRAWN FROM THE ACCIDENTS TO BOX GIRDER BRIDGES.

The linear theory of buckling of plates is based on the following main assumptions :

1) the plate is initially perfectly flat ;
2) the transverse displacement w at buckling are so small ($w/t \simeq 0{,}2$) that membrane stresses in the median plate are negligible.

Both these assumptions are wrong but, nevertheless, the linear theory can provide useful results *provided the safety factor* is selected on the basis of the relevant experiments.

The main practical outcome of above theory for stiffened rectangular plate panels is represented by both books by professor KLÖPPEL and his collaborators |A1| |A2|, in which the charts are restricted to simply supported plates reinforced by open section stiffeners. The author and his staff have generalized KLÖPPEL's mathematics for orthotropic plates reinforced by closed section stiffeners (|A4|, |A9|) and verified that this type of stiffeners are vastly superior to open section ones |A5|.

However, there is to-day a very large consensus that the linear buckling theory of plates is not an adequate support for the design of plate-and box girders (|A3|, |A9|).

The New-York IABSE Congress of 1968, in his conclusions, already stated |A10| that "The linear theory of plate stability is not an adequate basis for design of struts and girders consisting of thin-walled sections"....
"New carefully planned test series and computer simulation,"
including the post-buckling behavior (membrane action) of the sheet",
are therefore strongly recommended".

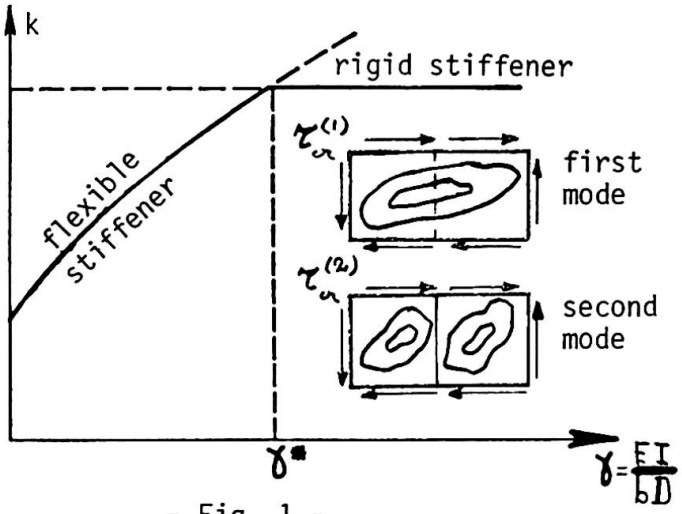
This opinion is based on the basic philosophy of the design rules which has been prevailing these last years. The philosophy generally accepted in Europe, for concrete as well as for steel, is the semi-probabilistic theory of limit sta-

tes,in which the adequacy of any structure must be controlled in two limit states:

a) the serviceability limit state (under working loads and other actions)
b) the ultimate strength limite state (under factored loads and other actions).

Now, if the linear buckling theory predicts accurately the bifurcation buckling load of a perfect plane panel, it does not tell anything about the postbuckling strength of this panel. This is dangerous because this postbuckling strength depends very much of the rigidity of the frame surrounding the plate and still more on the type of stressing (shear, bending or compression). Therefore, low safety factors derived from the shear case where it is known since the WAGNER studies |C2| that the strength reserve can be enormous, may be-and have been - misleading if they are applied to compressed panels.

There is another,even more dangerous defect of the linear buckling,which pertains to the concept of optimum relative rigidity (Mindeststeifigkeit) γ^* of the non-dimensional factor $\gamma = EI/bD$ (where EI is the flexural rigidity of the stiffener and $D = Et^3/12\ (1 - \nu^2)$ the rigidity of the plate). The linear theory predicts that such a stiffener has fifty chances on hundred to remain flat under the critical load (say τ_{cr}, fig. 1), and thus to enforce the second buckling mode (Mindesteifigkeit erster Art) or, at least, a mode where the stiffener remains *almost* flat (Mindesteifigkeit zweiter Art). As, theoretically, there is no benefit to increase γ beyond γ^* this notion has been extensively used (|A1|, |A2||A4|) to design stiffened plated structures. However, this concept has nothing to do with postbuckling strength. 22 years ago, the writer demonstrated experimentally |A3| that γ^* stiffeners invariably bent in the postbuckling range if not sooner and recommended to multiply the theoretical value γ^* by a coefficient m equal to 3 to 7 according to the position of the longitudinal stiffener, in order to obtain a stiffener quasi-straight up to collapse. He insisted that an increase of about 4 on γ meant roughly an increase of 2 on the cross section area, and that the additional expense was very small, because the price of stiffener depends very much more on fabrication, adjustment and welding than on its weight.



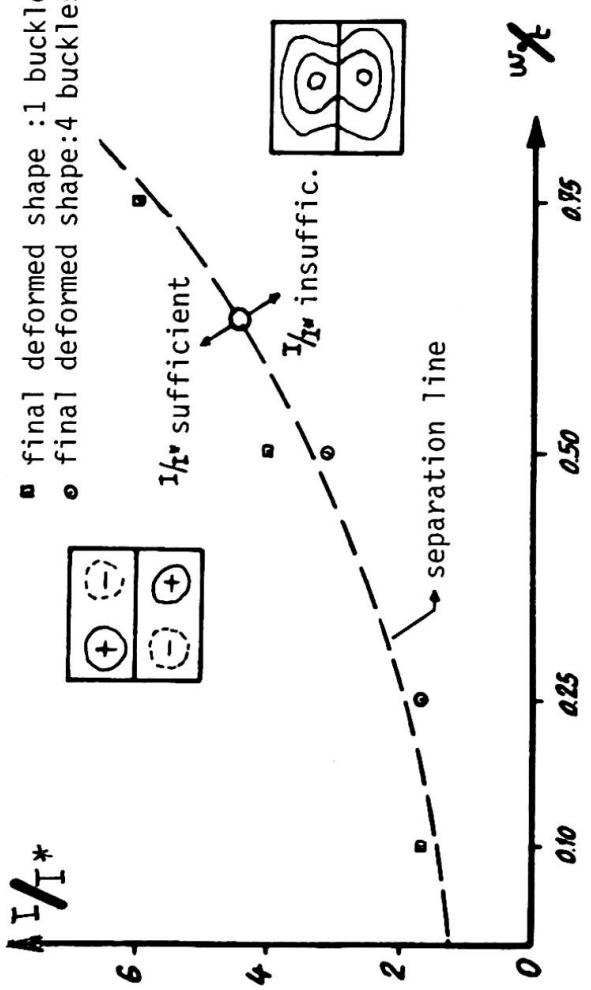
- Fig. 1 -

Professor P. DUBAS was the first to show, by experimenting on stiffened box girder models |D1|, that the γ^* concept was especially dangerous in the case of compression panels reinforced by many stiffeners.
The same result had been obtained theoretically by SKALOUD and NOVOTNY |A6| in the case of one or two stiffeners. Very recently, a collaborator of P. DUBAS, B. ROUVE, has obtained more striking results by using |A13| a large displacement finite element technique (see figures 2 and 3).

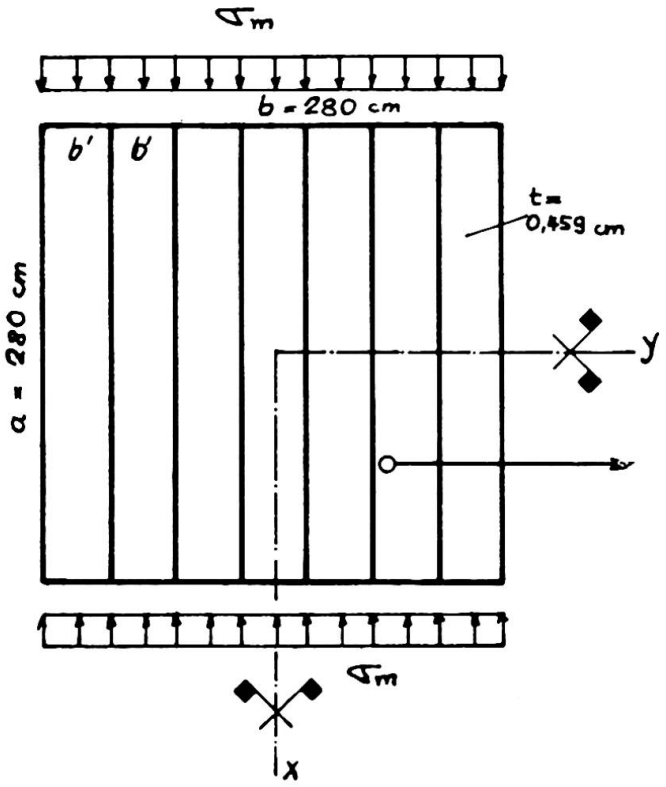
It begins to be possible, now, to derive the coefficient m arising in the relation

$$\gamma = m\,\gamma^*$$

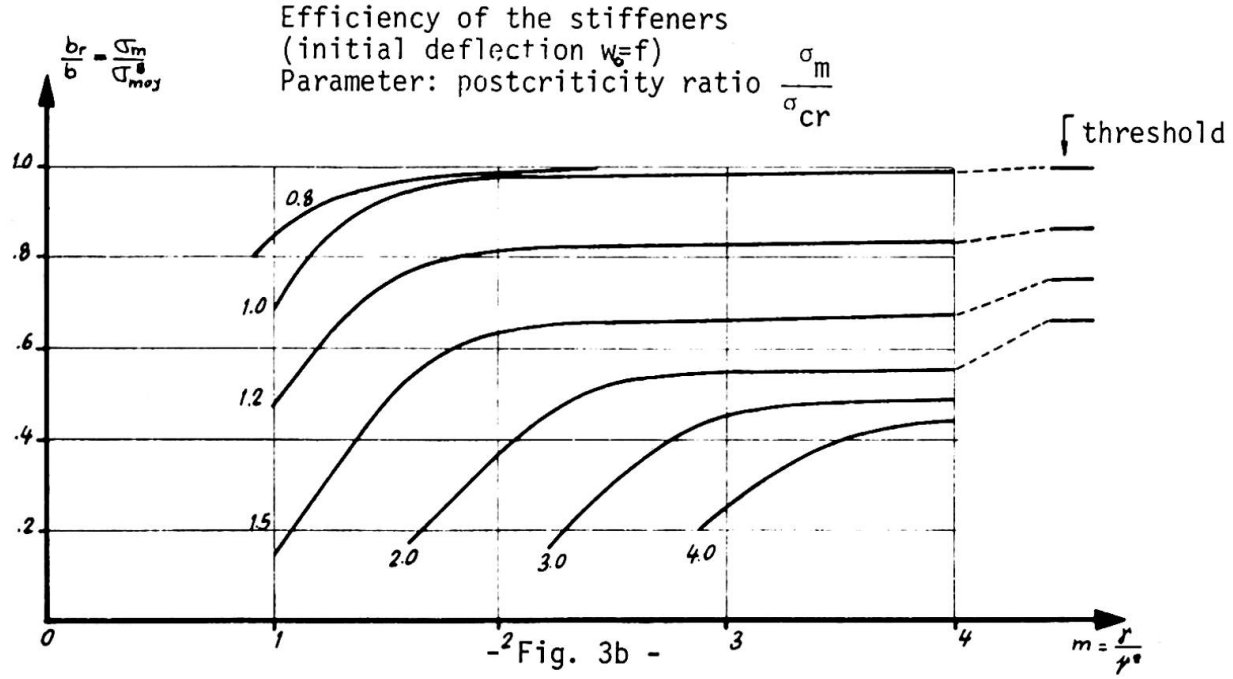
theoretically, as a function of the extent of the postbuckling domain in which the stiffeners are requested to remain quasi-straight, as well as of the initial imperfection of the panel considered.



- Fig. 2 -



- Fig. 3a -



- Fig. 3b -

This shows, in passing, the importance of a good quality of fabrication or, in other words, of close tolerances, not only on the geometrical imperfections, but also on the structural imperfections represented by the residual stresses. While some measurements on real structures have already been made |B10| and tolerances are given in many national specifications (e.g. |D.14|) no agreement could still be reached at the ECCS level within the subcommittee 8/3 (plate buckling) of ECCS. It is clear that tolerances very exacting and difficult to control, such as those proposed in the Merrison Rules |D4| can have a very deleterious effect on the price of fabrication and therefore hamper very much the development of steel bridges.

This is the more true because these rules impose the complete recording, in the factory, of the fabrication phase and a lot of corresponding measurements.

It is now generally acknowledged (|B3|,|B6|,|B7|,|B8|,|B9|) that the accidents that occurred under erection on four big box girders were due partly to poor detailing |B3|, and partly to inadequately low safety factors |D1||D2|, |B7|. Specialists differ about the relative importance of these two reasons. Personnally, I would say that, roughly, each of them bears half of the responsibility for the collapse.

It becomes clear, now, that the Merrison Rules, which had to be written in emergency conditions, are presently superseded;when they were published, they did represent a remarkable progress in many fields (shear lag, diaphragms,etc.). The engineering world was awaiting since two years the publication of a book that would explain clearly their scientific background. It seems, now, that this book will never appear.

It was clear, at the Conference held in July 1976 at Imperial College, London, that, due to the severity of these Rules, that are still in force in waiting for a new british code on steel bridges (the future BS 116 !), a very wide gap separates presently the fabricators and the research men who influence the design rules.

In the mind of present reporter, this is due to two very distinct reasons:

a) due to the advent of the limit state philosophy and therefore of the probabilistic interaction coefficients,the number of loading cases seems to increase to inacceptable limits.In London,Wex and Brown |D33| explained that, for a two-span continuous bridge, 1744 loading cases have in principle to be investigated. Even if more than 90 per cent of these cases may be immediately discarded on the basis of engineering judgment,it remains nevertheless that about ten different loading cases should normally be investigated ;

b) due to the introduction of the concept of *ultimate* strength limit concept, which replaces the concept of *elastic* response under *working* load, the analysis becomes now frequently elasto-plastic or, at least, non linear.

The duration and the price of the study of a box girder bridge are soaring up in such a way that some fabricators claimed in London for a dictator who would obtain the powers to simplify sufficiently the design rules.

To the present reporter, it is clear that the price to be optimized is the integrated price, comprising the study, and that therefore the length and difficulty of this study must be kept within reasonable limits.
It is equally clear that fabrication tolerances must be imposed, but they must be reasonable and simple to apply. The only way to reach this goal is to base them on extensive measurements on actual structures and to let fix them by a

commission composed, by equal parts, of research men, design specialists and fabrication and erection engineers, working hand in hand.

With these statements in mind, we shall now try to offer a general view of the present state of development of the ultimate design rules. It must be admitted that the research has been so intensive since 1970 that we are still in a very fluid state. The outcome of the studies undertaken in Western Germany is just beginning to appear and it seems that new-research will be launched in the U.S.A. under the sponsorship of the Subcommittee on Ultimate strength of box girders of the ASCE - AASHTO Task Committee on flexural members. Present report has therefore every chance to become obsolete in a foreseable future.

We shall first discuss plate girders and then, box girders. After that, we shall review some interim conventional design rules still based on the linear buckling theory. This report will conclude with brief considerations regarding the diaphragms of box girders, effect of local loads and other special problems.

3. ULTIMATE STRENGTH DESIGN OF PLATE GIRDERS.

Ultimate strength models for computing the ultimate carrying capacity of a stiffened panel in shear (plus eventual shear and compression) do not consider the linear plate buckling load as a limiting load, but as a basic value. Therefore, the design methods based on this concept are valid for technical construction with initial imperfections.

These methods all assume that the ultimate capacity can be evaluated from a cooperation of three independent actions, i.e.:
shear field-tension field-frame mechanism.
All methods assume that at ultimate strength, the shear field contributes the τ_{cr} stress of the linear theory, which this author feels to be a gratuitous assumption (see in this respect |C15|.) Some authors neglect completely the effect of the frame mechanism. The equivalent stress, calculated from the shear and the tension field, cannot exceed (under the factored loads !) the yield stress of the web material. When the frame mechanism is considered, one must take account of stresses induced by the bending moment of the girder by means of the bending rigidity and plastic moment of the flanges.

The basic model for ultimate shear strength of a plate panel is that developed between 1959 and 1961 at Lehigh University by K. BASLER (|C3|,|C4|, |C5|)under the guidance of B. THÜRLIMANN. This semi-diagonal tension field had already been studied by H. WAGNER in 1920 in the case of extremely thin webs (pure diagonal tension field |C2|) and it can be considered that a rather good approach of this basic model above was already given by RODE in 1916 |C1|.

The BASLER approach was criticized by several researchers(|C23|,|C6|,|C15|)on the ground of a computation error. This objection is not accepted by BASLER. Anyway, there are presently about ten different models for the ultimate strenth of a plate panel subjected to shear with eventual additional bending (|C8|,|C9| |C10|,|C11|,|C12|,|C13|,|C14|,|C16|,|C17|,|C20|,|C 21|,|C22|).

The corresponding theories, whose main characteristics are summarized in TABLE 1, are briefly described in the State of Art Report |C19| to which we are forced to refer, due to lack of space. These theories have various degrees of generality; the most general should be applicable to a plate panel

	Aspects	Aarau	Cardiff	Göteborg	Karlsruhe	Lehigh	Osaka	Prague-Cardiff	Stockholm	Tokyo	Zürich
1	Clarity of the method	fair	very good	very good	very good	good	fair	very good	very good	good	very good
2	Simplified method, easy for users	no	yes	yes	yes	no	no	no	yes	no	yes
3	Computer needed	no	charts	no	no	yes/no	yes	no/yes(1)	no	no	no
4	Account taken of longitudinal stiffeners	yes	yes	no	yes(2)	yes/no(3)	no	yes	no	no	no
5	Applicable to hybrid girders	yes	yes	yes	yes	yes	no	yes	yes	yes	yes
6	Applicable to unsymmetrical girders	yes	yes	yes	yes	yes	yes	no	no	yes	yes
7	Applicable to composite girders	no	nothing said	nothing said	yes	nothing said	no	no	no	nothing said	nothing said
8	Details for calculation of bending capacity	yes	yes	yes	yes	yes	no	yes	yes	yes	yes
9	Consideration of normal forces	no	no	yes	yes	no	no	yes	yes	no	yes
10	Details about σ_{cr} of compression flange	yes	mentioned	no	mentioned	yes	no	mentioned	mentioned	no	no
11	Pure shear	yes	yes	yes	yes	yes	yes	yes	yes	yes	yes
12	Normal stresses due to bending	yes	yes	yes	yes	yes	no	yes	yes	yes	yes
13	Combined shear and bending	yes	yes	yes	yes	yes	no	yes	yes	yes	yes

(1) for solving a cubic equation ;

(2) in the serviceability state ;

(3) computer method takes account of longitudinal stiffeners.

TABLE 1 - Summary of the aspects of the design methods.

dissymetrical (that is, with unequal steel flanges or with the flange replaced by a reinforced concrete deck), hybrid, reinforced by various transverse and longitudinal stiffeners and subjected simultaneously to internal resultants M, N, V (the N is important, e.g. for cable-stayed bridges). It can be safely said that none of the theories presently proposed does cover this whole field. On the other hand, because the problem is extremely complicated, these models are all based on different simplifying assumptions, so that someone has rightly remarked that there are as many models as members in the research committee. The author must mention, in addition, that, even if some theories are accompanied by simplifying charts obtained on computer and intended to design offices, it is difficult to avoid a "cooking recipe" type of presentation, with the corresponding danger of misunderstanding. At the last Colloquium of W.C. II of IABSE (Dresden, September 1975), the author has emphasized |A16| the danger that this type of rule may present for the profession. A clear physical insight of any design rules is absolutely necessary and it cannot be said, for example that the Merrison Rules |D4| are a model in this respect. Anyway, Working Group 8/3 of ECCS has undertaken the tedious task to calibrate the ten methods against *all* valid known tests. A similar calibration is contained in the American Guide about structural stability, edited by Bruce JOHNSTON |A19|.

The origin of the tests used is indicated below, while TABLE 2 gives, for each method, the mean value "m" of the ratio of predicted to observed load as well as the corresponding standard deviation.

Test results from different sources have been used to calibrate the design methods. These tests have been classified into two different groups :

(a) webs with transverse stiffeners only ;

(b) webs with transverse and longitudinal stiffeners.

Only the tests satisfying the requirements proper to each method have been considered for the statistical calculations (TABLE 2).

The sources are as follows :

BASLER, YEN, MÜLLER and THÜRLIMANN
ROCKEY and SKALOUD
LONGBOTTOM and HEYMAN
SAKAI, NISHINO and OKUMURA
EVANS, PORTER and ROCKEY
DIMITRI and OSTAPENKO
SCHUELLER and OSTAPENKO
SKALOUD
D'APICE and COOPER
COOPER, LEW and YEN
KONISHI
FUJII
CARSKADDAN
STEINHARDT and SCHRÖTER

Method	Transverse stiffeners only			Transverse and longitudinal stiffeners		
	n	m	sd	n	m	sd
Aarau	42	0.99	0.20	48	1,01	0,21
Cardiff	44	1.02	0.06	66	1.00	0.07
Göteborg	33	0.99	0.12	-	-	-
Karlsruhe	78	0.97	0.14	102	0.96	0.14
Lehigh	48	1.02	0.12	60	1.02	0.11
Osaka	36	1.07	0.17	-	-	-
Prague-Cardiff	32	0.98	0.08	46	0.99	0.07
Stockholm	33	0.96	0.11	-	-	-
Tokyo	40	0.95	0.15	-	-	-
Zürich	30/26	0.79/1.06	0.25/0.18	-	-	-

TABLE 2 Calibration against test results.

n : number of tests compared with design method ;

m : mean value of ratio of predicted to observed load ;

sd: standard deviation.

It is seen that, if some simple methods are deliberately safe, all can be used with confidence at least in the case of static loading provided they are not applied to exceptional cases which would be completely outside the field of the tests.

4. ULTIMATE STRENGTH DESIGN OF BOX GIRDERS.

While everybody agrees that there is a certain interaction between the webs and the compressed flange of a box girder, the complexity of the problem is such that, presently, all methods apply to isolated stiffened flanges subjected to a compression force either constant ($V=0$) or variable along the span ($V \neq 0$), plus eventually torsion ($M_t \neq 0$).

A problem still unsolved is whether it is economically interesting to "expand" the cross section in order to increase the strength of the girder, even at the expense of a reduction of the collapse stress, or not. A study of this problem would be especially welcome.

While computation models applicable to panels reinforced by very stocky stiffeners and used e.g. in naval architecture are known for a long time (see e.g. the state of the question made in |D2|), non linear models taking account of the actual support conditions of the plate panel (roughly simple support on the four edges), of the orthotropic character of the orthogonally stiffened panel and the membrane restraining stresses have appeared only very recently. We shall therefore divide the following discussion in two parts.

4.1. MODELS APPLICABLE TO STRONGLY STIFFENED PANELS, WHICH BUCKLE ALMOST CYLINDRICALLY.

After the YOSHIKI model |cf. D2| , the ultimate strength of stiffened panels could be evaluated by the FAULKNER formula, |cf. D2| , which, like the WINTER formula, is based on

the concept of effective width and which was derived from a series of full-size buckling tests of stiffened panels of warships.

This type of model is especially cultivated in Great-Britain, where it seems to be postulated that optimum design corresponds always to very stocky stiffening.

The models used for the design of stiffened plates were developed on the basis of several previous valuable studies of the non linear behaviour and ultimate strength of isolated panels, first in the elastic domain |D20|, then in the elasto-plastic one, with account taken of an initial curvature and of residual stresses (|D6|, |D7|,|D21|,|D22|,|D25|).

The first of this models, presented in the Merrison rules |D4|,was considerably criticized as being complicated and oversafe for what regards the serviceability criterion. The subsequent models tried to correct this main defect. They were developed by DWIGHT and LITTLE at Cambridge |D6|, HORNE and NARAYANAN at Manchester |D8|, WALKER and MURRAY at Melbourne |D7| and they have benefited from a considerable amount of tests, some of them on stiffened panels discarded from the original Lower Yarra bridge structure. Dr. DOWLING and his collaborators at Imperial College have developed a more general model |D31|, which is said to be able to take account of postbuckling effects, but has not been tested in the field of flexible stiffeners up to now. The degree of agreement of all those models with the tests will be discussed further on .

4.2. NON LINEAR MODELS APPLICABLE TO THE LARGE DEFORMATIONS OF RECTANGULAR PANELS SIMPLY SUPPORTED ALONG ALL FOUR EDGES.

Prof. DUBAS was the first to demonstrate experimentally |D3| that designs of compressed stiffened panels based on the γ^* star concept were highly dangerous and that the usual safety factor of 1.35, applicable to shear panels of plate girders, had to be considerably increased in the case of compression panels. MAQUOI, MASSONNET |A8| and SKALOUD |C7|showed that this conclusion could be derived theoretically in the case of stiffened imperfect panels reinforced by a unique longitudinal stiffener and uniformly compressed.

The Zürich design method proposed by Prof. DUBAS |D38| derives directly from his tests and is a modification of the classical (linear) buckling approach to stiffened plates.

As a measure of the stiffened plate's capacity to carry load beyond the critical buckling stress, use is made of MASSONNET's coefficient

$$m = \frac{\gamma_{post\text{-}cr}}{\gamma_{linear}}$$

where $\gamma = \frac{EI}{bD}$

is the well known relative rigidity of a stiffener.

DUBAS found by his tests that an empirical value of m = 4.5 gave a good agreement with the theoretically predicted failure loads. To estimate the moment carrying capacity of a given stiffened box girder, he proposes that :

1) The effective rigidity of the stiffeners be divided by five ;
2) the value of γ so obtained should be used (using eventually the charts of |A1, 2| to calculate σ_{cr} ;

3) the effective breadth of flange be calculated using σ_{cr} and hence the elastic section properties as obtained ;
4) the maximum moment be obtained using these section properties and $\sigma_{max} = \sigma_r$ (yield stress).

The first accurate evaluation of the postbuckling strength was obtained in 1971 by MAQUOI and MASSONNET's theory |D2|, which was subsequently shown to be in good agreement with an important series of tests (|D5|,|D9|,|D10|).

The theory is based on a suitable generalization of the von KARMAN-MARGUERRE non linear equations of moderate deflections of plates to orthotropic, eccentrically stiffened plates. The main assumptions are

a) The initial imperfection is a doubly sinusoïdal buckle (fig. 4)

b) The additional deflection is affine to the first buckling mode of the linear theory, which is also a doubly sinusoïdal buckle.

c) The boundary conditions of a plate panel situated between two successive nodal lines are those indicated in fig. 4.

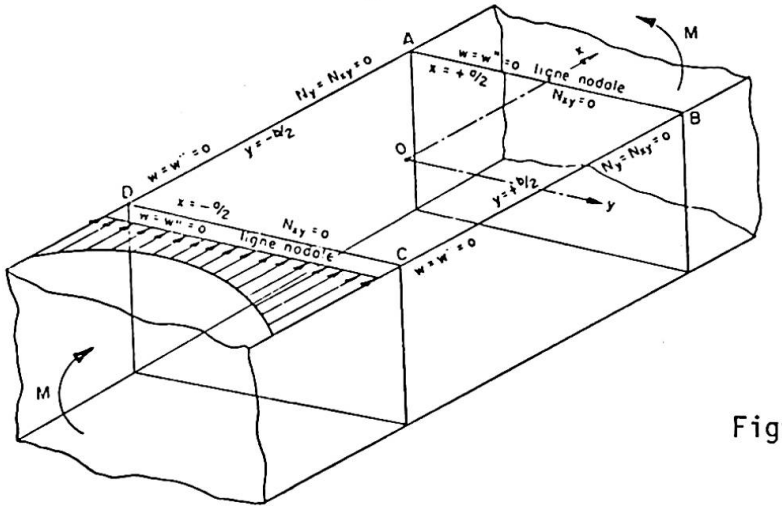


Figure 4

In the equations, the stiffeners are assumed to be "smeared" continuously on the compressed plate. The theory gives the efficiency ρ_g of such a "continuously stiffened plate". To take account of the discontinuous character of the stiffener, the effective width formula of FAULKNER is applied, which gives a local efficiency ρ_l. The total efficiency ρ_t is assumed to be the product of the two efficiencies ρ_g and ρ_l above.

The ultimate strength is assumed to be reached (in agreement with the recommendations of WOLMIR |A19|) when the mean value of the membrane stresses along the unloaded edges AD or BC of the compressed panel, reach the yield point. A series of large size models |D10| (Fig. 5) have been executed which have shown that

a) above criterion of collapse was accurate, but a little safe ;
b) the theory was in good agreement with the tests.
c) there was a considerable postbuckling strength reserve in the case of slender stiffeners.

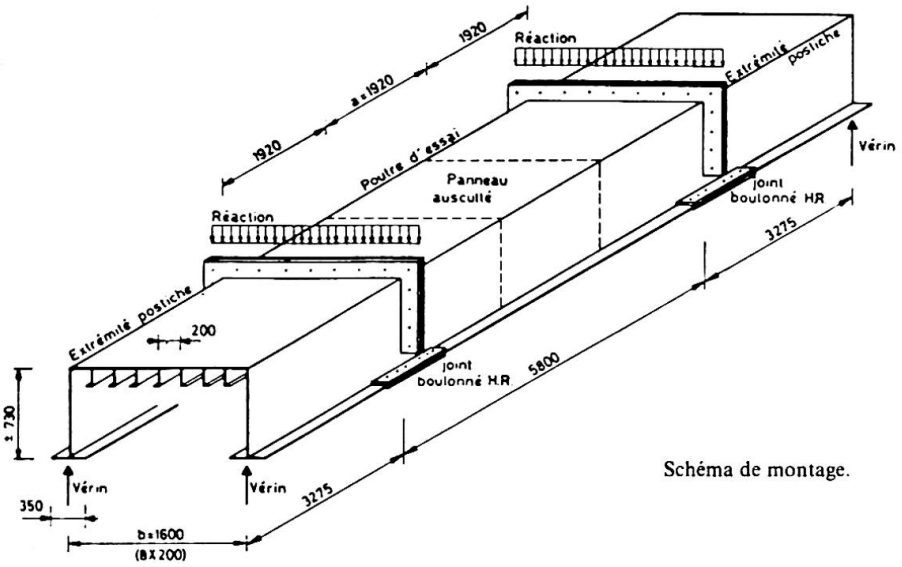
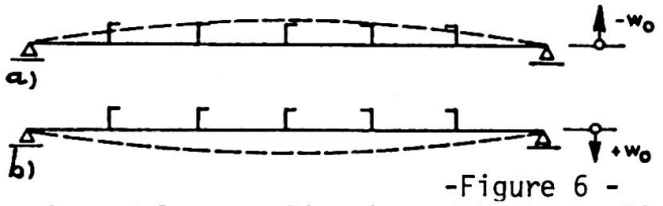


Schéma de montage.

- Figure 5 -

The Karlsruhe method developed more recently by STEINHARDT, VALTINAT and RUBIN, (|D17|,|D18|,|D32|) has several characteristics in common with the Liège method. However, it is more accurate, because it takes account of the discontinuous character of the stiffeners and leaves open the form of the transverse section of the buckle, which is determined so as to minimize the potential energy. It also distinguishes two types of collapse, by bending on the side of the stiffeners (fig. 6.a) and by bending on the side of the compressed plate (fig. 6.b).



-Figure 6 -

In addition, the thesis by Dr. RUBIN |D18| presents a series of charts that should facilitate considerably the practical use of the method.

A useful generalization of both non linear theories above has been presented in 1976 as Doktor Ingenieur Thesis by Dr.CHANG SUNG Pil |D19|. It is a non linear theory, based almost on the same assumptions as the Liège and Karlsruhe method. The ultimate strength N° 1 is the load for which the compression stress at the edge of the most stressed stiffener reaches the yield point. The ultimate strength N°2 is that for which, in the middle plate of the plane, the equivalent stress reaches the yield stress.

The main progress of the thesis is to be applicable also to closed section profiles. For open section stiffeners, it becomes almost identical to the MAQUOI-MASSONNET theory.

Large closed section stiffeners are now used quite generally for the orthotropic bridge floors. In his paper to the London Conference, DUBAS |D34| demonstrated that they furnish also a convenient and economical solution for longitudinal web and plate stiffeners. Intersection problems with the transverse stiffeners can be avoided by placing each category of stiffener on a different side of the plate. In his discussion of DUBAS's paper, KLEMENT (Austria) recommended highly the closed section stiffeners.
They are used in bridges in Austria since 20 to 30 years with no corrosion problems.

4.3. CHARACTERISTICS OF THE VARIOUS METHODS.

The degree of generality of the various methods may be seen from TABLE 3 in which the available methods have been separated in the two categories corresponding to sections 4.1. and 4.2. above, respectively.

Aspects of Stiffened Compression Flange, Covered in the Different Methods	Method							
	Merrison Rules	Cambridge Method	Monash Method	Manchester Method	Imperial College	Zürich Method	Liège Method	Karlsruhe Method
1. Asymmetry of cross-section of flange about horizontal axis	✓	✓	✓	✓	✓	-	-	✓
2. Buckling of plate between stiffeners	✓	✓	✓	✓	✓	✓	✓	✓
3. Torsional buckling of stiffener outstand	✓	-	-	-	✓	-	-	-
4. In-plane transverse stresses in flange plate	✓	-	-	-	-	-	-	-
5. In-plane shear stresses in flange plate	✓	✓	-	-	✓	-	-	-
6. Locally applied lateral loading	✓	-	-	-	✓	-	-	-
7. Variation in axial load along length	✓	✓	-	-	✓	-	✓	-
8. Overall curvature of box girder	✓	-	-	-	✓	-	-	-
9. Overall postbuckling behaviour of flange	✓	-	-	-	?✓	-	✓	✓
10. Welding residual stresses	✓	✓	-	✓	✓	-	-	-

TABLE 3

4.4. AGREEMENT BETWEEN THEORY OF TESTS.

The five british and australian methods are in satisfactory agreement with the corresponding tests, in which the lateral edges were kept free. To this reporter, these are wide column tests rather that tests on actual stiffened com-

pressed flanges, which are obviously held laterally (w=0) by the webs of the box girder.
For the details of the statistical comparison, which take two pages, the reader is referred to |D38|.

We shall reproduce here only the TABLE 4, which gives the statistical comparison between the 8 design methods and 12 actual tests on complete box girders, undertaken partly in Liège and partly in London. It is seen that all methods are safe - with varying degrees of safety - and give decent standard deviations, the more dispersed being the Karlsruhe and Monash Methods.

4.5. SHEAR LAG.

A problem that complicates the design of a box girder which is subjected to a bending moment M(x) variable along the span and accompanied therefore by the shear force V = dM/dx is that of shear lag. This expression designates the unequal distribution of the normal stresses over a cross section that is due to the (first order) in plane deformations of the component plates due to in-plane shear. This phenomenon was studied in the case of perfectly flat panels by ABDEL-SAYED |D13|. This problem was carefully studied at Imperial College by DOWLING and his collaborators, working for the Merrison Committee (|D11|,|D12|) and simple approximations for design have been obtained. They are more refined than the rules given by the german specifications about steel bridges |D14| because they take account of the orthotropic character of the compressed flange, or, otherwise speaking, of the effect on shear lag of the longitudinal stiffeners.

In the postbuckling range, there is another type of shear lag; MAQUOI and MASSONNET have shown (|D15|,|D27|) that, when the two types of shear lag occur simultaneously, the net effect is less than the product of the two isolated effects. In other words, a safe value of the efficiency of the girder is obtained by multiplying the efficiency ρ_t as given by the MAQUOI-MASSONNET theory in the case of pure bending (V = 0) by the efficiency factor ρ_o corresponding to the first-order shear lag of the perfectly flat panel.

In a very recent paper presented at the July 1976 london Conference, DOWLING, MOOLANI and FRIEZE, on the basis on three point-loaded box girder models |D28|, show that, in the ultimate condition, shear lag has little weakening effect on flange strength, due to plastic redistribution, and may be completely ignored if :

a) the stiffeners are stocky enough to sustain a strain equal to 2.5 the times yield strain without significantly unloading ;

b) the plate panels are capable of being strained up to 4 times the yield strain without significantly unloading.

The shear lag effects are also being studied now in Germany |D30|, but the researches are not enough advanced to produce definite conclusions.

The paper by LALLY and WOLCHUK |D29| gives interesting details about the american specifications for box girders bridges. It indicates that the design of plating in compression is based on an elastic-plastic buckling curve, which, like in Great-Britain, disregards any postbuckling effect.

Two still rather obscure problems are :
a) the design of the stiffeners for sufficient ultimate strength |D35| and
b) the behaviour of the compressed plate when (due to torsion and even shear-lag), it is subject to combined direct and shear in-plane loading |D36|.

DESIGN METHOD		TEST RESULTS		TOTAL	N
		LIEGE 6 tests	IMPERIAL COLLEGE 6 tests	12	
ZURICH	n	6	6	12	5
	m	0.869	1.004	0.937	
	sd	0.089	0.052	0.099	
MERRISON	n	6	6	12	1
	m	0.702	0.844	0.773	
	sd	0.058	0.138	0.126	
LIEGE	n	6	6	12	0
	m	0.751	0.878	0.815	
	sd	0.108	0.076	0.113	
CAMBRIDGE	n	6	6	12	0
	m	0.704	0.804	0.754	
	sd	0.063	0.048	0.075	
KARLSRUHE	n	6	6	12	1
	m	0.817	0.915	0.866	
	sd	0.107	0.178	0.170	
MONASH	n	6	6	12	0
	m	0.693	0.762	0.729	
	sd	0.168	0.110	0.100	
MANCHESTER	n	6	6	12	0
	m	0.756	0.871	0.814	
	sd	0.096	0.088	0.108	
IMPERIAL COLLEGE	n	6	6	12	0
	m	0.693	0.758	0.726	
	sd	0.086	0.123	0.111	

n = N° of tests compared with design method
m = Mean value of ratio of predicted to observed load
sd = Standard deviation
N = N° of cases where ratio of predicted to observed load exceeds one.

TABLE 4 - Calibration against box girder tests.

5. DIAPHRAGMS.

Not so long ago, diaphragms of box girders were verified by very crude methods. Because of the collapse at Milford Haven, which is attributed to a diaphragm weakness |B5| and due to the capability of elastic or elastoplastic finite elements, including eventually large displacements, a lot of studies have been devoted to diaphragms in Great-Britain. The first results, applicable to thick unstiffened diaphragms, are given in the Merrison Rules |B1| and studied in detail in |E2|. More recent results on stiffened diaphragms of trapezoïdal box girders have been published |E3| or presented in July 1976 at the London Conference (|E4| |E5|).

6. FABRICATION PROBLEMS.

The discussion at the London 1976 Conference have shown that it was absolutely necessary to establish a set of realistic fabrication tolerances that would be accepted together by the designers and fabricators. This would probably require an international cooperation.

We shall content ourselves, here, to mention the paper by HORNE and NARAYANAN |D24| in which the authors describe the effect on ultimate strength of welded stiffened panels loaded in longitudinal compression of various fabrication procedures (intermittent versus continuous plate/stiffener welds), various types of local imperfections, various procedures for restoring longitudinal straightness in panels which were bowed during fabrication. The perhaps most astonishing result pertains to the eccentric longitudinal stiffener splices on flat stiffeners, which had been heavily blamed after the Melbourne disaster (see |B6| to |B9|).

According to |D24|, these eccentric splices should be equivalent to concentric splices for what regards ultimate strength, provided a continuous connection exists between stiffener and plate.

7. OTHER PROBLEMS OF PLATE GIRDERS : PLATE GIRDERS WITHOUT STIFFENERS, EFFECTS OF CONCENTRATED LOADS, HYBRID GIRDERS, SPECIAL PROBLEMS.

There are a lot of special problems concerning plate and box girders which should be now discussed. The first of them concerns the effect of more or less concentrated transverse loads applied to the compression flange and acting on the girder in combination with the general resultants M, V, N. This problem seems to-day almost clarified, due to the efforts of ROCKEY |C7| |F4| and BERGFELT |F5|. The swedish school of research has also proposed, since about 15 years, a set of rules for the design of plate girders of moderate size, used in buildings, and subject to distributed static loads. These girders have no stiffeners, except at the supports. The main progresses in this field have been obtained by GRANHOLM, BERGFELT |F1| and HÖGLUND |F2| . HÖGLUND has also given rules |F3| to take account of the presence of holes in the web.
The swedish rules are based on the postbuckling resistance of the thin web.
Commission 11 of ECCS, under the chairmanship of Prof. REINITZHUBER, has actively studied the problem and ECCS Recommendations for the design of this type of girder will be published very soon.

At the London Conference of July 1976, van DOUWEN has demonstrated |F6| the technical and commercial advantages of this type of girders, which enable the fabricator to reduce considerably his investment in plates compared to the considerable stock which must be maintained in the case of rolled profiles.

8. THE PRO- AND AGAINST ULTIMATE STRENGTH DESIGN RULES FOR PLATE AND BOX GIRDERS.

While ultimate strength design rules are in principle vastly superior to the conventional rules based on the linear theory of buckling, there are numerous objections against them, which eventually carry a lot of weight.

8.1.

In principle, - as was emphasized in the London discussions at ICSPS by professor STEINHARDT, structures subject to moving loads should be designed elastically (even if advantage is taken of non linear membrane effects) and real plastic design should be reserved to compact sections, in the AISC Specifications meaning of this word. The author has always defended this point of view; indeed, not to speak of the fatigue problems, there seems to be a blunt contradiction between designing sections for ultimate strength and still continuing the determine the M,N,V diagrams on continuous girders by using elastic theory; the fact that a similar situation exists in reinforced and prestressed structures is not sufficient to justify it.

8.2.

Above limitation of ultimate strength methods to bridges can be obtained automatically if proper control is made of the serviceability limit state. We think, here, of the "breathing" of webs, which can produce fatigue cracks in the heat-affected zones near the flanges and/or transverse stiffeners (|A11|, |A12|).

It seems that our tchecoslovakian colleagues have similar ideas, because the paper by DJUBEK and SKALOUD |C18| about the new tchecoslovakian standard for plate girders distinguishes carefully between fixed and moving loads. In the case of moving loads, non linear elastic rules, due to DJUBEK, are proposed, while plastic mechanisms (ROCKEY-SKALOUD) are reserved for the case of fixed loads. According to DJUBEK, Professor BROUDE has developed similar rules in USSR for the design under moving loads.

The ultimate concession regarding plastic action that could be made to ultimate strength methods would be to take as ultimate load the shake-down load (in the sense of this expression in plastic theory) and not the collapse load. Unfortunately, this would still complicate further the design rules.

8.3.

The elasto-plastic phenomena at collapse being extremely intricate, the ultimate strength methods are forced to rely on simplified models and assumptions. While at least some of these models are conceptually simple, the design rules take easily the shape of "cooking recipes", that the designer does not understand, what the present author considers to be productive of misunderstanding and therefore highly dangerous for the profession, not to

speak of the unpleasantness of the teaching |A16|.

8.4.

The ultimate strength methods cannot be general. New technical developments about cable-stayed bridges, (biaxial compression plus shear in the foot of the towers, etc) offshore structures, etc... are continuously bringing about new problems that must be solved without waiting until the proper corresponding ultimate strength model is developed.

8.5.

For all these reasons, that have been analyzed a little differently by SCHEER and NÖLKE |A18| at the London Conference on Plated Structures, there is a need for interim design rules, still based on the linear buckling theory, because the designers need them to build.

These rules |A16| have been developed in cooperation between the german Working Group "Plate Buckling" of the german "Unterausschuss für Stabilität" and the Working Group 8/3 of ECCS, and particularly Prof. DUBAS, Dr. MAQUOI and the author. They have appeared as an Appendix to the ECCS Recommendations for the Design of Steel Structures and are justified scientifically in the Manual on Structural Stability |A15|.

For the detailing problems (welded or bolted connections of plates and stiffeners), above rules give detailed prescriptions which should prevent the return of collapses during erection.

The safety has been adjusted by the introduction of correction factors so as to take account of the varying amount of postbuckling strength with the type of loading of the panel (shear or compression) as demonstrated by the tests. While it is openly admitted that this "manipulation" of the safety factors is not a very scientific procedure and that the safety should rather be a result of the basic principles of the design theory, it is felt that the resulting rules have the considerable advantage of being both simple and safe. The author believes therefore that, in spite of their title "provisional rules", they will still be applied for a long time.

9. RECOMMENDATIONS AND CONCLUSIONS.

9.1.

An extremely large research effort has been made these last six years towards a better understanding of the ultimate carrying capacity of plate and box girders, but it remains to digest the corresponding research papers in order to transform them into acceptable design rules.

9.2.

Quite apart from the particular problem of plate and box girders, the semi-probabilistic theory of limit states should be carefully weighted in order to maintain the computational effort of the designer within a decent percentage of the total price of the structure. For this to be obtained, methods should be found to identify the few limit states that are really governing.

9.3.

The Merrison Rules were interim rules, written in a period of emergency. They have performed their main duty, which was to protect against new accidents. The research launched by the Merrison Committee has brought a wealth of new scientific information.

9.4.

The Merrison Rules must now be replaced, as soon as possible, by more simple specifications, because they are hampering the development of long span steel bridges. The truth is somewhat in the middle between the Merrison Rules and the dangerous oversimplified rules that some designers would favour at any cost. It will, however, be difficult to obtain these desirable balanced rules by Committee work, because research men and fabricators have widely different views. Perhaps will it be necessary to appoint a kind of wise and competent "dictator" to "distill" the enormous amount of research now available.

9.5.

A set of realistic and easy to control tolerances should be established within a committee comprising by equal parts research men, high format designers and fabricators-erectors. IABSE and ECCS should be helpful in discussing this problem at the european or even worldwide level. The control of these tolerances could be restricted to a certain sampling (5 % ?) to be agreed upon by the parties. Anyway, continuous recording of measurements on all parts of a bridge at the factory during fabrication is inacceptable.

9.6.

Studies should be continued to investigate whether elastic shear lag (and possibly also stresses arising from restraint of warping are "eroded" by plastic action. These studies should comprise fatigue - or, at least, repeated loading - tests, in order to assess to which extent the conclusions apply to bridges.

9.7.

Systematic comparative computations should be undertaken to assess whether optimization requests the stocky stiffeners commonly used (especially in Great-Britain) or whether it is economical to "expand" the cross section, as explained in detail in the beginning of section 4.

9.8.

It would be extremely instructive to compare on a wide basis these realistic ultimate strength design rules that we are still awaiting with the provisional rules based on the conventional theory evoked in section 8.5., in order to see whether the increase in scientific accuracy is justified by a sufficient increase in safety and or by an economy of the total price of the structure (that is the price including the design expenses).

9.9.

Summarizing, the main question mark still is: To which extent are we entitled to take benefit of plastic redistribution in the case of bridges supporting moving loads ?

10. ACKNOWLEDGMENTS.

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SUMMARY

After a brief summary of the tendency of research these last years, which is towards the development of ultimate strength, the author examines critically the linear buckling theory and emphasizes its shortcomings. Then, he reviews briefly the ultimate strength models presented for plate and box girders and compares their prediction with the full set of available experiments. After a brief review of the problems concerning shear lag, diaphragms and fabrication problems, he discusses the pro- and against ultimate design rules and shows the interest of previsional rules still based on the linear theory suitably adapted.

RESUME

L'auteur présente un bref résumé de l'évolution de la recherche au cours des dernières années, qui tend vers le développement des théories de la résistance à la rupture, puis il examine de façon critique la théorie linéaire du voilement et souligne ses faiblesses. Il rappelle brièvement les modèles de résistance à la rupture utilisés pour les poutres à âme pleine et en caisson et compare leurs résultats avec l'ensemble des expériences réalisées. Après un bref résumé des problèmes de "shear lag", de "diaphragme" et de fabrication, l'auteur considère les avantages et les inconvénients des règles de dimensionnement basées sur la résistance à la rupture; il souligne l'intérêt de règles provisoires basées sur la théorie linéaire adaptée de façon adéquate.

ZUSAMMENFASSUNG

Nach einer kurzen Darstellung der Forschungsentwicklung der letzten Jahre, die in Richtung von Traglasttheorien tendiert, unterzieht der Autor die lineare Beultheorie einer kritischen Prüfung und unterstreicht ihre Mängel. Traglastmodelle für Vollwand- und Kastenträger werden besprochen und die entsprechenden Ergebnisse mit allen vorliegenden Versuchen verglichen. Nach einer kurzen Zusammenfassung der Probleme der verminderten Mitwirkung infolge Schubverzehrung, der Querscheiben und der Anfertigung erörtert der Autor die Vor- und Nachteile von Traglast-Bemessungsvorschriften und stellt fest, dass provisorische Vorschriften, die auf einer entsprechend angepassten linearen Beultheorie beruhen, von Interesse sind.

Progress on Bridge Loading

Progrès dans la charge des ponts

Fortschritte bei der Erfassung der Belastung von Brücken

WILLIAM HENDERSON
Vice President of IABSE
Chartered Civil and Structural Engineer
Edinburgh, Scotland

Report on the Proceedings of a Colloquium on Highway Bridge Loading. Cambridge, 1–10 April 1975 Organised by the British Group IABSE

General

Thirty six papers were contributed to the colloquium. The procedure adopted was that all papers were circulated to participants well before the date of the colloquium so that they could be read and digested, enabling the participants to be ready and prepared for the discussions at Cambridge. The procedure was eminently successful; all papers were in the hands of participants in adequate time and were clearly studied very thoroughly.

The various papers are individually of such value that it has been decided not to summarise them in the published report. They are available in full from the British Group. A list of the papers and authors is appended.

This report is a substantially abbreviated version of a much fuller report which is also available from the British Group. It attempts to review the substance and general conclusions of the very active discussions which took place over three days of concentrated application by the participants. It contains no extracts from the submitted papers for the reason given and does not attribute any statements to individuals since its object is to attempt to present a consensus of opinion. The author accepts full responsibility for his interpretation of this consensus.

Very considerable thanks and appreciation are due to the participants who produced much valuable work and devoted themselves to a very strenuous and intensive discussion.

All countries appear to have similar problems in respect of the classes of traffic which use their highways. Broadly, traffic can be classified into three groups.

(a) Normal vehicles designed to conform to the general legal limits imposed in each country. These normal vehicle weights range in maxima up to some 400kN gross laden weight. This weight is fairly typical of most countries. In Ontario, Canada, however the permitted laden weight can be as much as 700 kN.

(b) Extra Normal vehicles, typified by engineering construction equipment, the gross weight of which ranges up to some 1,500kN. The movement of this type of traffic is generally subject to some degree of control by law but enforcement of the controls appears in most countries to be fairly limited in its efficiency. There is, therefore, with these vehicles a not inconsiderable risk of them travelling on routes where it is undesirable they should be.

(c) Abnormal Vehicles, which are special vehicles designed to carry indivisible loads of considerable weight such as electricity transformers and other heavy electrical plant. Loads up to some 6,000kN are to be considered in this group for which movement is in general closely controlled in most countries.

Whatever the form in which vehicular loading is presented for bridge design purposes it is a model and should be clearly recognised as such by those who apply it. The weight and disposition of vehicles varies from instant to instant and from bridge to bridge; the loading model must embrace the most severe effects of well defined categories of vehicles.

The view was taken that there should be one design loading model to represent traffic of categories (a) and (b) and another model to represent vehicles in category (c).

One of the defects of some design loading models in use in the past has been that while they catered for normal traffic reasonably well they did not do so satisfactorily for extra normal traffic; for medium and longer span bridges the main longitudinal members could sustain quite large overload effects due to these category (b) vehicles but short secondary members were severely limited. It was considered that any satisfactory traffic loading model for general application should be contrived to eliminate this defect and to produce a reasonably consistent overload capacity in all elements of a bridge.

With very few exceptions the view was taken that the most satisfactory form of model is a uniformly distributed load associated with one or two point loads. The point load or loads are intended to provide the necessary adjustment required to make it possible for one uniformly distributed load to be used for both bending and shear on the smaller elements of the bridge structures and also, of course, for the shorter span bridges.

Due to the mix of traffic, ranging from private cars (generally by far the largest number of vehicles using a highway) up through several gradations to the heaviest truck to be considered (generally a quite small proportion of the traffic) consideration must be given to the attenuation of the vehicular loading model as the loaded length becomes greater. On a short loaded length it is a reasonable expectation that it can be filled with closely packed vehicles of the heaviest type. As the loaded length increases the interspersion of lighter vehicles will reduce the equivalent intensity. The characteristic load is not sensitive to the nature and mix of the traffic for short and medium loaded lengths but does become very sensitive to these as the loaded length increases.

It is also the case that as the width of carriageway increases in terms of the number of traffic lanes available so too will the characteristic load decrease across the carriageway.

Several views were offered on how the loading model should be constructed to take these phenomena into account in respect of longitudinal attenuation;

(A) That the intensity of the distributed load in a traffic lane should be taken as constant over short and medium loaded lengths and thereafter diminished in a smooth curve as the loaded length increases.

(B) That a constant distributed load occupying a discreet loaded length of around 30 m should be taken and a series of these loaded lengths considered with gaps of no live loading between, the gaps progressively increasing in length as the total overall length under consideration increased.

The important consideration to be kept in view is that whatever device is adopted, the severe intensities associated with short and medium span structures must also be expected to occur on long structures and to occupy the same loaded length as is appropriate to the medium span bridge. These high intensities must be considered as occupying that part of the influence line with the largest ordinates.

While this is self evident when applying an arrangement of distributed load such as is described in (B), it can and should be applied with the arrangement described in (A) also.

In respect of transverse attenuation two devices were again presented;

(C) That one or two lanes should be considered to be occupied by the full intensity of uniformly distributed load appropriate to the loaded length concerned and that all remaining lanes should be considered to be occupied by a stipulated fraction of the appropriate full intensity (say 1/3).

(D) That the effect of multiplicity of lanes in the width of the bridge should be regarded simply as an extension of the treatment of the attenuation of uniformly distributed load in relation to the longitudinal loaded length. In applying this device the longitudinal lengths of each lane would be summed and the total load taken on the bridge would be represented by the product of this total loaded length and the appropriate intensity of distributed load associated with it. Where, however, elements of the structure were severely affected by shorter loaded lengths as, for example the length of one lane or part of a lane, the intensity of load appropriate to this particular loaded length would be applied within it and the remainder of the total for the whole bridge allocated to other lanes.

It remains to be established whether or not the patterns and disposition of traffic in adjacent lanes of a multi-lane highway are affected in the same way as is the case for increasing lengths of any single lane.

Although, the prevailing viewwas that a uniformly distributed load with one or two point loads provided the most satisfactory model there was, nevertheless, a strongly held conviction of a few participants that for short and medium loaded lengths it was most satisfactory to specify a pattern of trucks, reserving the distributed load concept to long loaded lengths.

Methods of Derivation of Loading Models

In the submitted papers there is a number of most excellent studies of statistical techniques and information presented. It remains, however, that there is much yet to do in the way of collection of data related to traffic weights and disposition on the highways and the formulation of trends oftraffic behaviour. Among the aspects which call for study is the formation of convoys of heavy trucks, both as a consequence of the "properties" of different types of vehicle and as a result of the deliberate decision by organisations or drivers. By "properties" is meant relative operating speeds, manoeuvrability and other characteristics. It will no doubt also be necessary to extend the studies to take into consideration the "properties" of different kinds of highways. Studies are also desirable of traffic behaviour in special circumstances (accident, unexpected road block, throttling of traffic by unexpected reduction in the number of lanes at some point in an otherwise wide and free running carriageway). Basically there is also the need to collect data concerning the weights and spacings of axles and their sequence in different lanes of carriageways, taking into account that trucks are frequently not carrying their maximum capacity in terms of weight although they may be in terms of volume.

It was generally agreed that while probabilistic studies based on thoroughly researched data were of vital importance the derivation of a loading model could not be based on these alone. The final form and intensities of the traffic loading model would have to be based on the "credibility" approach supported by the probabilistic studies. These studies would contribute towards the important objects of;

1. Achieving a consistent standard of safety.

2. Ultimately establishing a quantified level of risk.

3. Reinforcing confidence in the adequacy of the loading model.

One of the serious difficulties in using the "credibility" approach is that on short loaded lengths where it is clearly feasible that a tightly packed group of the heaviest trucks could be present, the only way in which excess loading could be occasioned would be the overloading of vehicles. For this there is little room for doubt, but on long loaded lengths where the intensity of loading is diminished to reflect the attenuation of traffic it becomes possible to speculate on situations where the traffic might consist

of an exceptionally large number of the heaviest trucks which might become closely packed because of some blockage, while it might be that some of these trucks were grossly overloaded and these might unfortunately occupy the region where the influence line ordinates are largest. While this statement clearly is grossly exaggerated it nevertheless points up the contribution which probabilistic studies can make to (3) above.

Assuming that loading models are already in use, as they are in most countries, the development of well founded probabilistic studies related to various loaded lengths should make it possible to re-shape the model accordingly as the level of risk has been over or underestimated at different regions, thus achieving the objectives (1) or (2) above.

In this respect it is worth noting that for the small or moderate span bridge where the loading model is likely to be most realistic, quite large changes in design loads will have only minor effects on cost. On the other hand in large span bridges where the greatest uncertainty is, and the tendency is that the loading model is likely to have been made unduly severe, the cost of providing the strength to carry excessive traffic loading becomes substantial.

The problem remains at the present time, when the amount of statistical work done falls so far short of the desirable, how should an acceptable level of risk be established? The problem is not made simpler by the adoption of limit state design philosophy. It was considered that no serious attempt should be made to quantify the probabilities in numbers, but rather that the development should be made from experience of live load models used now and in the past. Are existing bridges safe for the levels of traffic we have today? If so, this state should be preserved. Comparisons can be made by equating the design loading effects when applied to established "permissible working stress" methods with the effects when applied to "limit state" methods and a suitable load factor derived. Since former methods generally used the same factor for dead loads as for traffic loads, while the latter method introduces low load factors for dead load, where there is a greater degree of certainty about its amount, it follows that the two methods can only be equated at one selected ratio of dead to live load and this must be chosen as a satisfactorily low value.

Vehicle regulation in law and changes in the future

A satisfactory standard of law enforcement in respect of the laden weights permitted on heavy trucks was considered essential, as was also the control of the weights and movements of these vehicles classified as extra normal and as abnormal.

The permitted weights of vehicles and the safe carrying capacity of bridges are intimately associated. It seems remarkable therefore that few states appear to use their highway bridge traffic loading model directly as a datum which also controls what may be permitted as maximum laden weights in their vehicle regulations. Only in Great Britain and Ontario it appears is there a deliberate formulation of permitted vehicle weights directly in terms of bridge loading.

In discussing the allied problem of the extent to which the loading model should anticipate future trends in the development of vehicles in terms of permitted weight it was generally conceded that this was something which could not be done in any developed country on a rational basis although some attempts had been made to do so on a short term basis to the extent of around 10%. The

fact is surely that any developed country has such a great capital investment in existing bridges that it is not conceivable that all of these should be made inadequate by a decision to introduce substantially greater carrying capacities.

In developing countries which presently may have comparatively light traffic it is possible to predict with a degree of confidence that traffic weights will trend towards those of developed industrialised societies and it is considered that the capital investment being made in new bridges should take this into account. Development and industrialisation can be seriously inhibited by inadequate bridge capacity.

Category C Vehicles All industrialised countries have the problem of moving exceptionally heavy, generally indivisible loads of industrial, frequently electrical equipment such as transformers, generators, large presses, even ships propellors, rudders, stern frames and the like. The needs of different countries tend to differ in accordance with the prevailing industrial activities and the existence within the country of other forms of transport such as railways and waterways capable of carrying these loads.

The extent to which highway bridges should be constructed to cope with these is very much a matter of the economics of each country. Few, if any of these loads are indivisible in the absolute sense, but simply are more cheaply and satisfactorily built in some factory equipped to construct them economically. To what extent bridges and routes should be designed to carry such loads is a matter of balancing the economics of constructing an adequate route against these other considerations, giving due account to the considerable nuisance these loads are to other highway users.

The number of these vehicles in any country is not large and they are generally of much the same configuration. Because of this the most appropriate model to be used for bridge design is an idealised vehicle. The movement of these vehicles on the roads is normally very strictly controlled; given the extent to which this control is reliable it is open to decide the extent of normal traffic which ought to be considered to be with these loads on bridges.

Dynamic Effects

Braking Forces There appears to be a very wide variation in the specified requirements of various countries. It was considered that this was an area where a review of these requirements was desirable. The increased adhesion possible between road surface and vehicle, the increased speeds of heavy vehicles and the use of elastomeric bridge bearings and piers which flex to accommodate temperature movements of the deck make it desirable that adequate braking forces should be specified. Two aspects of traffic should be considered, the first being several heavy trucks following each other at a speed appropriate to them and spaced out as they would be at that speed. An emergency application of brakes in the first truck would be followed by a similar application successively in the following trucks so that over a calculated period of time several trucks would exert the maximum braking force simultaneously. The second aspect would be the collision of one fast moving heavy truck with some other stationary but substantial truck or other object which might be disposed in such a way that progressive collapse of the latter would not dissipate a large proportion of energy.

Centrifugal Forces For much the same reasons as for braking forces a review of these was desirable. The succession of heavy trucks suggested for that effect would also be an appropriate condition to be considered for centrifugal forces.

Impact Effects Defining these as the outcome of dropping a mass on to a surface and also the sudden application of mass to an element, the former could vary quite considerably from country to country, depending on the standards of road construction and maintenance.

Little study appears to have been done on the effect of sudden application of loads to elements with very short loaded lengths, some of which may in fact have a cantilevered form where a wheel may exert its maximum effect at the instant of its first contact with the element (plates at expansion joints are an example); further investigations into this aspect are desirable.

Vibration and Damping There was general agreement that impact loading and dynamic response, though closely related should be treated separately so far as the effects on a bridge and its elements were concerned. The specification of a standard impact factor together with a maximum permitted deflection per unit of span is not considered to be an adequate or satisfactory check for the dynamic response of a bridge.

The calculation of the natural frequency should constitute part of the normal design procedure. Suitable approximate methods are given in papers submitted. Dynamic response can lead to the equivalent of an increment in vertical load which has been treated as part of the impact factor but preferably should be considered separately; where the vibration mode of the element is below 4.5Hz the value of this vertical increment of load can be substantially increased.

Vibration is of particular significance on footbridges where discomfort and alarm may be caused to the user. Tolerance levels are discussed in submitted papers.

The damping capacity of bridge structures has not been adequately researched and knowledge in this area is severely limited. Further work in this subject is strongly recommended and should be rewarding.

Fatigue Dealing first with stress cycles due to the dynamic response to vehicles, surveys of heavy vehicle behaviour have shewn that with normal traffic flow there is little interaction due to the pressure of several vehicles. One stress cycle per vehicle with a 6% amplification of the static stress (giving a 1.2 factor on damage) would provide a reasonable basis in this respect.

The random variable stress cycles due to the passage of traffic can be related to design strengths derived from sinusoidal constant amplitude data using either Miner's cumulative damage rule or the Root mean square-stress range relationship. Both methods are dealt with in submitted papers.

Load Spectra should be based on observed national traffic and vehicle load conditions. These require to be extensively and thoroughly studied and reliable observations made of vehicle spacings, range of proportions of maximum load carried, alignment of successive vehicles behind each other and so forth.

It is believed that there is a strong tendency to make cumulative overestimates of the stress ranges when making assessments a priori from limited data. The view was expressed that the load spectrum should vary in accordance with the locale of the bridge and that it should vary in accordance with the span.

The fatigue threshold is not well defined under high cycle (10 to 100 x 10^6) variable and constant amplitudes and further research is desirable in this area. So too is the need to develop techniques in fatigue monitoring and crack detection having practical application to bridges. Since the pattern of variable stress cycles and the number of cycles cannot be predicted with any degree of precision the same applies to the prediction of the initiation of cracks. Currently available monitoring devices (magnetic and acoustic instruments) have not been effective and painting which is essential to prevent corrosion obscures any fine cracks effectively, thus it is unlikely that any crack will be visually observable until it has opened quite significantly. It was agreed that some sort of displacement induced stress indicator should be developed towards this object and installed in steel bridge structures.

It was emphasised that strength in fatigue relied essentially on good detailing. Specifications should be framed to encourage designers to use good details and stringent quality control should be exercised at all stages during fabrication and erection. With these provisos it was general felt that fatigue was probably not such a serious problem as it was thought to be by many authorities.

So far as corrosion fatigue is concerned it was considered that in normal environments with reasonable maintenance this was not a matter for concern.

The question of the design life of bridges was discussed. Several countries do not appear to specify this. It is, of course, necessary in dealing with fatigue in order to assess the number of cycles of stress to be considered. It is also essential in limit state design in order to establish partial factors which are time related. Some participants were, however, of the opinion that for the latter purpose the "life" could be taken as substantially less than the planned life of the bridge.

Collisions with Piers and Parapets Useful contributions were made on this subject. These deal with the control and containment of light vehicles such as private cars and also heavy trucks, but a great deal more work and study is required in the latter field. Calculation of the resistance to collision using dynamic methods related to energy dissipation is the most attractive approach to providing realistic and flexible solutions for specific situations as opposed to the rather arbitrary requirement of a strength to withstand a static force.

Closing Ceremony

Cérémonie de clôture

Schlusszeremonie

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Address

Allocution

Rede

FRITZ LEONHARDT
Prof. Dr.–Ing.
Chairman, Technical Committee of IABSE
Stuttgart, GFR

Mr. President of the Japanese Organizing Committee -
Mr. President of I A B S E -
Ladies and Gentlemen -

The Japanese Organizing Committee has asked me - as Chairman of the Technical Committee - to give a report at this Closing Ceremony. Let me first say:

> The 10th Congress of our Association in Tokyo was just marvellous, and I am sure that all participants had been impressed deeply.

The pre-congress publications, Introductory Report and Preliminary Report, contained already many valuable papers to the themes which were chosen. The lively interest of the engineers from all over the world in the problems and solutions put before us, was well displayed by the many different contributions in the prepared and free discussions. The amount and range of new and valuable information is so large that it is impossible for me to give a just appraisal or summary of the results to any of the different themes. I shall try, however, to collect my general impressions and draw from them some recommendations for our future work:

Most deeply I was impressed by the amount and quality of studies and research work which has been done during the last decade by Japanese engineers. I am sure that all foreign participants agree when I express my gratitude to our Japanese colleagues for making available so valuable results of their work to our profession by their numerous contributions. Research in our structural engineering field is throughout the world on a good path by combining the theoretical approach with testing models and measuring strains or deformations at built structures. Our task is to find truth, this means to know the real response of our materials not only in laboratory specimens but also in the structures themselves. Many interesting examples of such testing have been described. The techniques of model testing and measurements at structures have considerably improved and some highly specialized institutes in this field dispose of valuable experience. It is desirable that this special know-how will be discussed and exchanged in a symposium in the near future.

Theories are often based on idealized assumptions and these assumptions must be checked by such test results. Only those theories which are based on true

physical data can be fully approved for practical design. Such data are often complex and complicated but the computer allows us to consider them correctly. Several contributions - e. g. in the field of fire resistance - demonstrated such good use of computer aid.

In the field of computer-aided design optimization I am not too optimistic about the soundness of the proposed methods. To design a structure or a bridge is a process of creativity, even of intuition based on a very wide range of knowledge in many aspects. The computer has no creativity, it cannot change or vary the conceptual design idea or develop an improved fabrication or construction method. Such computer-aided optimization programs may even be a handicap for the creative thinking of the design engineer, leading him into too narrow channels and limiting the horizon of his thoughts, of his vision and fantasy.

Being in Japan, we should study the methods of the century-old schools of Japanese art, how to become a master in his art: It is by concentrated studies of all the basic knowledge of this art, by learning all about the tools which are necessary to exert this art and by practising the use of these tools until they are available without the need to think of them so that the mind can concentrate on the creative act in a synthetic way almost in the state of subconsciousness.

This perfection of creative art is too difficult to be reached in engineering. There are too many aspects and too many different fields of science which are our tools, which must be considered and rational thinking cannot be left behind. But we can learn from Japanese art that the use of knowledge and methods must be well trained that we must govern our tools and aim at the synthesis. The difficulty is that the quantity of the necessary knowledge is so very large and that the tools to use the knowledge are also manyfold so that a single individual can almost not collect all what is needed for a good design. This can only be overcome by teamwork, by the good will to respectful cooperation. Such cooperation should also be fostered with those who are in charge of inspection and maintenance. There had been some contributions pointing to this necessity to learn of damages under the impact of service and weather. Such experience must be fed back to the design offices. We have too many engineers who drill a deep hole for finding some small knowledge, which we need, but his drilling took so much of his mental capacity that he lost the survey. We need more engineers who can collect results and can judge interdependences and can assemble all these analysing work to a good sythesis.

In order to reach this aim, we must stop to make everything more and more complicated, we must stop to see scientific glory in getting so sophisticated, that the average engineer can no more understand. It was Mr. Dicke who told us a fine story ending with the request to keep things simple. The highest level of art is simplicity.

This Association plans to publish surveys, state-of-art reports, in order to help engineers to get again a wider range of knowledge and tools for their design work. These survey reports must also be kept simple and understandable. I think that we are already on the way in this direction and that many of you will join us in these efforts.

Our structures must fulfil functional, social and economic requirements. Social requirements include invironmental aspects. If we look at structures which had been built during the last decades, then we find only few which please us by their appearance in the environment. Real beauties, which would enjoy us like Japanese ladies in their beautiful kimonoes, are very rare. So we should give more emphasis to aesthetic features of our design work. We engineers should not leave that to the architects alone, there should be a good cooperation which requires that engineers themselves study the rules of aesthetics. There are national commissions at work for this task. Our Association plans also to get active in this field and I would appreciate help from our members. We plan to have a session on aesthetics of engineering structures at our next Congress in Vienna.

This Association has also the aim to promote international cooperation and understanding. International exchange of newly gained knowledge and practical experience becomes more and more important; it can help to save a lot of national money and to improve the service which our work shall give to society.

One of the barriers to improve this cooperation is the language problem. It can be overcome if we follow the principle that everybody should be able to speak his mother tongue and English. This Congress has shown us again that we need such a simplification. I would much like to ask the many university professors who are attending this Congress from countries other than English speaking, that they may take influence on their educational authorities to have English as primary foreign language. They should also send their assistants and as many students as possible to English speaking countries. But let us get effective courses in good pronounciation! In about twenty years we should have such congresses with one language only. But in the cultural field, each nation should preserve and cultivate her national costums, art and specialities to avoid annoying uniformity.

The President of the Japanese Organizing Committee, Mr. Yoshihiro Inayama, mentioned in his speach at the opening ceremony that engineers should help to build a peaceful world and avoid war. We all emphatically agree. Engineers can be proud of the great progress they have achieved in improving the living conditions and welfare of mankind. We have learned also that technical progress itself is not sufficient to satisfy society. Progress must be controlled by ethics, by assuming the moral and ethic responsibility for how technical and scientific progress is used.

Modern war is misuse of technical achievements, it cannot solve problems but causes only suffering and destruction. I believe that it is in the power of us engineers to stop misuse of the fruits of our work. It is our d u t y to fight against such misuse. We should refuse to do anything which in our knowledge leads to destruction and suffering. Peace can be strengthened if natural scientists and engineers join in solidarity and refuse to do harmful work. Our profession's aim and duty must be to serve society and mankind for improvement of life,and avoid any destruction. In this sense, this Congress was a Congress for Peace.

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Closing Address

Allocution finale

Schlusswort

TAKEO NAKA
Vice-Chairman
Japanese Organizing Committee
Japan

Mr. President Professor Cosandey, Ladies and Gentlemen:

Now we are approaching to the time of closing the 10th Congress of IABSE held in Tokyo, 1976. I should say many thanks to members of the Executive Committee, the Permanent Committee, the Technical Committee, Working Commissions, Task Groups, Sessions Chairmen, Technical Advisors, General Reporters, Contributors, Discussors and Simultaneous Interpreters. Although our staffs of J.O.C. have made their much efforts to this congress, something troubles or deficiencies on the process of session-meetings have occurred during these days, as you know, however the Congress seems to be successful due to the kind collaborations and good wills of all participants from 40 countries. I express my heartfelt regards to these generous participants more than 1,500 in number.

Members of IABSE in Japan and J.O.C. have to tell their most gratitude to the Executive and Permanent Committee of IABSE for the honour of the First International Award which was given to Professor-Emeritus Dr. Kiyoshi Mutō of the University of Tokyo for his contribution for a half century on the Earthquake Engineering on the Structures, various kinds of Vessels and Nuclear Reactors.

Moreover I would like to inform you that our Chairman of J.O.C., Mr. Inayama signed to accept that the symbol mark of the 10th Congress of Tokyo will be permanently used for the Symbol of IABSE. This symbol mark is originally designed from the simplest form of structure and the ancient Chinese character -- HITO -- which means human being. There are various view points about this symbol. I can take it as a bridge to tie countries of the world and also a home where people can rest. I am willing to present the flag with this symbol to President Professor Cosandey at this closing ceremony.

The next congress will be held in Vienna, Austria in 1980. I hope all participants to the 10th Congress will attend at the 11th Congress and will keep their mutual familiarity and exchanging their technical and scientific knowledges for the development answering the purposes of IABSE.

Thank you.

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Discours final

Schlussansprache

Final Speech

MAURICE COSANDEY
Professeur
Président de l'AIPC
Lausanne, Suisse

Monsieur le Président du Comité d'Organisation,
Monsieur le Président du groupe japonais de notre Association,
Mesdames et Messieurs,

Notre congrès se termine par un double succès: celui de la préparation et celui de l'intérêt des discussions qui ont été menées au cours de nos séances. Mais il y a également un succès indirect, si je puis dire. C'est celui des entretiens informels, hors séances de travail, et des rencontres fructueuses, soit pour faire avancer un point encore obscur de notre art, soit pour préparer l'avenir. Ces rencontres informelles sont d'une importance capitale dans l'établissement du bilan d'un congrès. Je puis dire quant à moi, et sans vouloir faire de l'autosatisfaction, que le bilan du congrès 1976 de l'AIPC à Tokyo est extrêmement positif.

M. le Professeur Leonhardt vous a fait tout à l'heure un premier rapport de synthèse sur les résultats scientifiques et techniques du congrès. Je voudrais, pour ma part, insister pour l'avenir sur l'intérêt, pour les auteurs et pour leurs auditeurs, d'une insertion de chaque contribution dans le cadre du thème traité. Je m'explique. Dans l'intérêt de la créativité des auteurs, un thème, tout en étant très bien défini, doit avoir une ampleur suffisante, car nous sommes une association de personnalités aux connaissances et à l'expérience larges. Dans ces conditions, il est indispensable qu'un exposé particulier soit placé dans le contexte du connu du thème auquel il se rapporte. Ayant eu le privilège de travailler de nombreuses années dans l'industrie, avant de me consacrer à l'enseignement et à la recherche universitaire, j'ai toujours apprécié, dans une publication technique, les conditions relatives aux comparaisons avec d'autres idées, méthodes ou conceptions. D'une nouvelle méthode de calcul, il est intéressant de connaître les différences qu'elle comporte par rapport aux méthodes en vigueur, et ceci soit sur le plan du résultat économique, de la durée du travail ou simplement de son élégance. Des considérations au sujet du pourquoi de l'utilisation de tel ou tel matériau ne manqueraient pas, très souvent, de pertinence. Notre Association a pour mission de toucher tous les matériaux. Un effort plus grand peut et doit être fait à cet égard lors de nos symposiums et congrès.

Mesdames et Messieurs, j'ose espérer que notre congrès a réalisé deux buts implicites qui, dans mon esprit, ont toujours été très importants. Celui de renforcer l'amitié entre nos peuples et celui de faire un grand pas dans la compréhension de l'âme de ce pays et de ses habitants.

Jeudi, une partie d'entre nous a eu le privilège de parcourir le pays et d'admirer la splendeur et la richesse de la nature. Celles et ceux qui ont fait le voyage pour voir le Fuji-San sont retournés déçus à Tokyo, car le Fuji ne s'est pas montré. Au XVIIe siècle, le poète Basho s'est aussi rendu (mais à cheval et à pied) sur place pour effectuer un pélerinage au Fuji. Il ne l'a pas vu en raison d'un épais brouillard. Est-ce parce qu'il était à pied et qu'il a pu jouir par l'odeur, l'ouïe et la vue de la splendeur de la nature qu'il s'est écrié "Séki koyuru hiwa hinémosu améfurité yama wamina kumoni kakurétari kirishiguré fujiiominuhizo omoshiroki" - "Belle est également cette journée grise où la pluie dérobe le Fuji", ou est-ce un état d'âme que nous aurions intérêt à cultiver ? Je vous laisse méditer la réponse.

L'organisation d'un congrès exige une multitude de bonnes volontés, un génie du management et une dose sérieuse de bonne humeur et de volonté. Le Comité d'organisation a su réunir toutes ces qualités et je le félicite en notre nom et au nom de l'Association. Il ne m'est pas possible de remercier nommément chaque personne ou société qui ont contribué ici à Tokyo au succès du congrès. Je me permets, cependant, de présenter à M. Yoshihiro Inayama, président, à l'intention du Comité tout entier, notre admiration et notre reconnaissance pour tout le travail accompli. Nous associons à cet hommage de gratitude les associations ou sociétés suivantes:

Science Council of Japan
Architectural Council of Japan
Japan Society of Civil Engineers
Japan Road Association
Society of Steel Construction of Japan.

Nous devons accroître notre rayonnement, non pas comme un but en soi, mais parce que nous croyons rendre service à la communauté internationale. Pour faciliter nos relations avec tous les intéressés, nous avons cherché depuis quelques temps un sigle, afin de marquer régulièrement notre présence. Ce sigle, nous l'avons trouvé grâce à nos amis japonais qui ont inventé, pour notre congrès, le "toit à deux pans que vous connaissez. Ce sigle du dixième congrès à Tokyo devient dès aujourd'hui le sigle de l'Association Internationale des Ponts et Charpentes, et nous remercions le Comité d'organisation de le mettre avec leur amitié à notre disposition.

Personne ne peut vivre sans une philosophie qui peut être une éthique, une religion, une idéologie, une morale ou autre chose encore. Dans la philosophie japonaise, il existait et il existe peut-être encore les sept "dieux du bonheur". L'un est Hotei, le dieu de la largeur d'esprit et de la richesse. J'ai compris ici le mot richesse dans le sens de culture étendue, de connaissances assimilées, d'abondance d'imagination créatrice, sans vouloir mépriser, bien sûr, ce qui est nécessaire pour déguster un sukiyaki. C'est la grande leçon que je retirerai de mon séjour au Japon, c'est la largeur d'esprit et la richesse de l'âme de ses habitants.

Nos meilleurs voeux accompagnent la famille impériale, le gouvernement et le peuple japonais.