12th CONGRESS
12e CONGRÈS
12. KONGRESS

VANCOUVER, BC

Sep 3 -7, 1984

INTRODUCTORY REPORT
RAPPORT INTRODUCTIF
EINFÜHRUNGSBERICHT

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Association Internationale des Ponts et Charpentes AIPC
Internationale Vereinigung für Brückenbau und Hochbau IVBH

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Preface

The Canadian Group of the IABSE together with the Canadian Society for Civil Engineering have invited our Association to hold the 12th Congress in Vancouver, B.C., Canada, from September 3 to 7, 1984. The general theme chosen for the Congress is "Structural Engineering Today and Tomorrow". We feel that Vancouver is a perfectly fitting place for this theme. Situated in a splendid geographical location it is a fast developing modern city in the Pacific region. It is in this part of the world that major economical and industrial activities can be expected. They in turn will require inventive sturctural engineering to build the necessary facilities.

A Scientific Committee under the chairmanship of Dr. R.A. Dorton has prepared the technical programme. The present Introductory Report contains the lectures defining and surveying the different themes. It serves also as a call for papers. We hope that many members and other Congress participants will respond to this invitation and submit valuable contributions. Besides Plenary Sessions and Seminars the Technical Programme offers Poster Sessions, a Film Programme, a Technical Exhibition and Technical Tours.

A National Committee with Prof. P.F. Adams as chairman as well as a Local Organizing Committee with Mr. D.H. Jamieson as chairman have been responsible for the organizational arrangements. To give our Congress a well rounded human touch an informative complementary cultural and social programme will be offered. Together with our Canadian colleagues, we look forward to greeting many of our members and other professional colleagues with their spouses and friends in Vancouver in 1984.

Zurich, August 1983

Prof. Bruno Thürlimann
President of IABSE

Préface

Le Groupement Canadien de l'AIPC en collaboration avec la Société Canadienne de Génie Civil ont invité notre Association à tenir son 12e Congrès à Vancouver, BC, Canada du 3 au 7 septembre 1984. Le thème général retenu pour le Congrès est "Génie des structures — aujourd'hui et demain". Nous croyons que Vancouver est la place idoine pour traiter de ce thème. Situé dans une région splendide, au bord du Pacifique, Vancouver est une cité moderne et qui évolue rapidement. Elle se trouve dans cette partie du monde où d'importants développements économiques et industriels sont attendus. Des solutions nouvelles dans le génie des structures permettront de répondre à ces besoins.

Le programme technique du congrès est préparé par un comité scientifique sous la direction de M. R.A. Dorton. Le présent Rapport Introductif comprend des exposés précis et synthétiques sur les différents thèmes. Il sert également d'invitation et d'appel aux communications. Nous espérons que de nombreux membres et également d'autres participants au Congrès soumettront des travaux intéressants aux différentes séances. Le programme technique comprend, outre les séances plénières et séminaires, des poster sessions, un programme de films, une exposition technique et une excursion technique.

Un comité national d'organisation, avec le professeur P.F. Adams, président, ainsi qu'un comité local d'organisation, sous la direction de M. D.H. Jamieson, sont responsables des problèmes d'organisation. Des manifestations culturelles et sociales sont également prévues pour donner à notre Congrès un cadre digne et amical. Avec nos hôtes canadiens, nous espérons que de nombreux membres et autres collègues de la profession — accompagnés de leurs familles — participeront au Congrès de Vancouver en 1984.

Zurich, août 1983

Prof. Bruno Thürlimann
Président de l'AIPC

Vorwort

Die kanadische Gruppe der IVBH zusammen mit der Canadian Society for Civil Engineering haben unsere Vereinigung eingeladen, den 12. Kongress in Vancouver, B.C., Kanada, vom 3. bis zum 7. September 1984 durchzuführen. "Konstruktiver Ingenieurbau — Heute und morgen" wurde zum Generalthema des Kongresses gewählt. Wir sind der Ansicht, dass Vancouver eine Stadt ist, welche für unser Thema beispielhaft ist. Die in einer geographisch wunderschönen Umgebung in der pazifischen Region gelegene moderne Stadt befindet sich in einer raschen Entwicklung. In diesem Teil der Erde können wichtige ökonomische und industrielle Fortschritte erwartet werden. Zur Bewältigung der dazu notwendigen Bauaufgaben sind neue innovative Lösungen zu finden.

Das technische Programm des Kongresses wird von einer wissenschaftlichen Kommission unter der Leitung von Herrn Dr. R.A. Dorton ausgearbeitet. Der vorliegende Einführungsbericht enthält die Vorträge, welche die verschiedenen Themen näher definieren und zusammenfassend darstellen. Er dient gleichzeitig als Einladung, Beiträge für den Kongress einzureichen. Wir hoffen, dass viele Mitglieder und auch weitere Kongressteilnehmer interessante Arbeiten zu den verschiedenen Themen einreichen werden. Das technische Programm umfasst neben den Plenarsitzungen und Seminarien auch "Poster Sessions", Film-Vorführungen, eine technische Ausstellung und technische Exkursionen.

Die organisatorische Vorbereitung des Kongresses wird von einem Nationalen Komitee unter der Leitung von Herrn Prof. P.F. Adams und einem lokalen Organisationskomitee unter der Leitung von Herrn D.H. Jamieson durchgeführt. Zusätzliche kulturelle und gesellschaftliche Anlässe sind geplant, um dem Kongress einen würdigen menschlichen Rahmen zu geben. Mit unseren kanadischen Gastgebern hoffen wir, viele Mitglieder und weitere Teilnehmer mit ihren Gemahlinnen und Freunden in Vancouver in 1984 begrüssen zu können.

Zürich, im August 1983

Prof. Dr. Bruno Thürlimann
Präsident der IVBH

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INTRODUCTION

1. PREAMBLE

The aim of this introduction is to give some information about the working sessions of the Congress. Information relating to the Poster Sessions, the technical excursions, the film programme, the exhibition, as well as other points, appear in the Final Invitation, published in December 1983.

Sixteen themes have been chosen for the 12th Congress of IABSE to be held in Vancouver, BC, from 3rd to 7th September, 1984. This Introductory Report attempts to define and describe these themes in order to serve as a basis for contributions from members of IABSE and others interested in participating in the Congress ("call for papers").

2. SCOPE OF THE CONGRESS

2.1 Daily Themes and Top Lectures

Each day of the Congress from Tuesday 4th to Friday 7th September, 1984 is introduced by two Top Lectures allocated to the daily themes as follows:

Sept. 4th A Structural Design Process

Prof. Ben Gerwick, San Francisco, CA, USA
Mr. Jean Muller, Clichy, France

Sept. 5th B Engineering and Construction Management

Prof. Angelo Pozzi, Baden, Switzerland
Mr. Hiroatsu Takahashi, Tokyo, Japan

Sept. 6th C Structural Engineering in Extreme Environments

Prof. Luis Estava, Mexico City, Mexico (tentative)
Mr. V.N. Saphonov, Moscow, USSR

Sept. 7th D New Frontiers in Structural Engineering

Prof. Edmund Happold, Bath, England
Dr. T.Y. Lin, San Francisco, CA, USA

Top lectures will be presented in the main hall from 9 to 10 a.m. with simultaneous translation. There will be no concurrent session.

2.2 Plenary Sessions

Daily themes are discussed further in Plenary Sessions in the main hall from 10.30 to 12.30 h with simultaneous translation.
Working Sessions for Daily Themes A and B will have presentations by invited speakers. Time will be allowed for free discussion.
Plenary Sessions for Daily Themes C and D will have presentations of selected contributions from a call for papers and will be followed by free discussions.

2.3 Seminars

Simultaneously with Plenary Sessions in the morning, and also in the afternoon, Seminars will be conducted presenting selected contributions from a call for papers and will be followed by free discussions. Seminars are conducted without simultaneous translation with the exception of Seminars IV, VIII, XII.

The following Seminars are scheduled:

I	Health ans Safety in Construction
II	Computer Aided Structural Engineering
III	Transit Guideway Structures
IV	Thermal Performance of Buildings
V	Developments in the Design of Steel Structures
VI	Wind Effects on Structures
VII	Developments in the Construction of Steel Structures
VIII	Snow and Ice Effects on Structures
IX	Developments in the Design of Reinforced and Prestressed Concrete Structures
X	Developments in the Design and Construction of Wood Structures
XI	Developments in the Construction of Reinforced and Prestressed Concrete Structures
XII	Topic of Concern in 1984

All Seminars are subject to a "call for papers", with the exception of Seminar XII.

The "Topic of Concern in 1984" (Seminar XII) was not decided at the time of editing the Introductory Report (July 1983), so as to reserve the possibility to present a theme of immediate concern at the Congress.

2.4 Poster Sessions

The following themes are also dealt with in the form of Poster Sessions:

- Structural Design Process
- Health and Safety in Construction
- Computer Aided Structural Engineering
- Innovative Structures
- Transit Guideway Structures
- Thermal Performance of Buildings
- Engineering and Construction Management
- Structural Engineering in Arctic Regions
- Structural Engineering in Earthquake Zones

The Poster Session has become customary at IABSE Congresses. Unlike the presentation of papers in plenary sessions, the Poster Session is more akin to an exhibition, where small groups of people especially interested in a particular subject gather around the author of an exhibition panel. This proven type of session leads to deeper and more personal discussions. Instructions for the preparation of posters will be drawn up by the IABSE Secretariat.

Posters may be proposed to any of the themes indicated under 2.4. The Posters on a Congress theme will be grouped in an area and for a period of time which will be indicated in the Congress programme (usually for a whole day). The authors will, according to the schedule to be determined, make themselves available by their posters.

2.5 Congress Programme

The Congress Programme to be handed out to the participants together with further Congress documents will contain the time schedule of lectures and contributions as well as their summaries. It will enable the participants to choose which session to attend and arrange their own programme.

3. CONTRIBUTIONS

3.1 Invitation for Submitting Contributions

With the exception of the Daily Themes A and B, prepared contributions by attendants are welcome to any of the Plenary Session, Seminar and Poster Session.

Participants wishing to submit contributions are requested to send to the IABSE Secretariat before the 1st December 1983, the title, the theme concerned as well as a one page summary in one of the three official languages of the Association. The contribution must be directly related to the theme as treated in the Introductory Report. Authors who would prefer to present their report in the poster session are requested so to indicate on the title page. The Scientific Committee will make a first choice in January 1984.

3.2 Selection of Contributions

According to the tentative decision on the summary, the authors will be requested to send in their full text of the contributions set out according to IABSE guidelines to the Scientific Committee before April 10, 1984. The Scientific Committee will be responsible for the selection of contributions. Authors will be advised of the committee's decision by May 10, 1984.

3.3 Publication of Contributions

The Final Report will be published after the Congress and will contain, in addition to the Top Lectures, the contributions and posters to all Congress themes, a selection of the free discussion as well as the conclusions of the General Reporters.

The Final Report will be sent to all the congress participants after the Congress.

INTRODUCTION

1. PRELIMINAIRE

Cette introduction a pour but de donner quelques renseignements sur les séances de travail du Congrès. Des renseignements sur les Poster sessions, les excursions techniques, le programme de films, l'exposition ainsi que d'autres indications apparaissent dans l'Invitation Finale publiée en décembre 1983.

Seize thèmes ont été retenus pour le 12e Congrès de l'AIPC à Vancouver, BC, du 3 au 7 septembre 1984. Le Rapport Introductif a pour but de définir et de décrire ces thèmes afin de servir d'appel aux communications des membres de l'AIPC et des personnes intéressées à participer au Congrès.

2. DEROULEMENT DU CONGRES

2.1 Thèmes quotidiens et exposés magistraux

Chaque journée du Congrès, du matin 4 au vendredi 7 septembre 1984, commence par deux exposés magistraux relatifs au thème quotidien:

4 sept. A Le processus du projet

Prof. Ben Gerwick, San Francisco, CA, USA
M. Jean Muller, Clichy, France

5 sept. B Gestion du projet et de la construction

Prof. Angelo Pozzi, Baden, Suisse
M. Hiroatsu Takahashi, Tokyo, Japon

6 sept. C Structures de génie civil dans des conditions extrêmes

Prof. Luis Estava, Mexico City, Mexico (provisoire)
M. Vladimir Saphonov, Moscou, URSS

7 sept. D Nouvelles frontières du génie des structures

Prof. Edmund Happold, Bath, Angleterre
Dr. T.Y. Lin, San Francisco, CA, USA

Les exposés sont présentés, avec traduction simultanée, dans la salle principale, de 9 h à 10 h. Il n'y a pas de séances parallèles à ce moment-là.

2.2 Séances plénières

Les thèmes quotidiens sont approfondis dans des séances plénières, avec traduction simultanée, dans la salle principale, de 10.30 à 12.30 h.

Les séances plénières des thèmes quotidiens A et B font l'objet de conférences présentées par des orateurs invités. Une discussion libre est prévue à la suite de ces conférences.
Les séances plénières des thèmes quotidiens C et D offrent une sélection de contributions, sur la base d'un appel aux communications. Une discussion libre s'ensuit.

2.3 Séminaires

En même temps que les séances plénières du matin, et également l'après-midi, des séminaires présentent une sélection de contributions, sur la base d'un appel aux communications. Des discussions libres s'ensuivent. Les séminaires sont conduits sans traduction simultanée, à l'exception des séminaires IV, VIII, XII; ces trois séminaires ont la traduction simultanée.

Les séminaires suivants sont organisés:

I	Prévention des accidents dans la construction
II	Génie des structures assisté par ordinateur
III	Structures des moyens de transport en site propre
IV	Comportement thermique des bâtiments
V	Développements dans le projet et le calcul de constructions métalliques
VI	Effets du vent sur les structures
VII	Développements dans l'exécution de constructions métalliques
VIII	Effets de la neige et de la glace sur les structures
IX	Développements dans le projet de constructions en béton armé et précontraint
X	Développements dans la construction en bois: projet, calcul et exécution
XI	Développements dans l'exécution de constructions en béton armé et précontraint
XII	Sujet d'actualité en 1984

Tous les séminaires font l'objet d'un appel aux communications, à l'exception du séminaire XII.

Le "Sujet d'actualité en 1984" (séminaire XII) n'était pas décidé au moment de la rédaction du Rapport Introductif (juillet 1983), afin de réserver la possibilité de présenter un thème d'intérêt immédiat, lors du Congrès.

2.4 Poster sessions

Les thèmes suivants font également l'objet d'une présentation sous forme de Poster sessions:

- Le processus du projet
- Prévention des accidents dans la construction
- Génie des structures assisté par ordinateur
- Structures nouvelles
- Structures des moyens de transport en site propre
- Comportement thermique des bâtiments
- Gestion du projet et de la construction
- Structures de génie civil dans les régions arctiques
- Structures de génie civil en zones sismiques

La Poster session n'est plus une inconnue aux Congrès de l'AIPC. Différant des exposés en séance plénière, la Poster session ressemble à une exposition où de petits groupes de personnes intéressées à un sujet particulier, se retrouvent autour de l'auteur d'un panneau d'exposition. Ce genre de séance, qui a fait ses preuves, conduit à des discussions plus approfondies et plus personnelles. Des directives pour la préparation des posters sont établies par le Secrétariat de l'AIPC.

Des posters peuvent être soumis pour chacun des thèmes indiqués sous 2.4. Les posters relatifs à un thème du Congrès sont réunis dans un endroit et pour une période déterminée, selon le programme du Congrès (normalement un jour entier). Les auteurs sont présents devant leurs posters selon un horaire déterminé.

2.5 Programme du Congrès

Le programme du Congrès, remis aux participants en même temps que d'autres documents du Congrès, contient l'horaire des conférences et des contributions, de même que leurs résumés. Les participants peuvent ainsi choisir les séances qui les intéressent et ainsi organiser leur propre programme.

3. CONTRIBUTIONS

3.1 Appel aux communications

A l'exception des thèmes quotidiens A et B, des contributions préparées par les participants sont les bienvenues aux séances plénières, séminaires et Poster sessions.

Les participants souhaitant soumettre une contribution sont invités à envoyer au secrétariat de l'AIPC, avant le 1er décembre 1983, le titre, le thème concerné ainsi qu'un résumé d'une page dans l'une des trois langues officielles de l'Association. La contribution doit avoir une relation directe avec le thème traité dans le Rapport Introductif. Les auteurs préférant présenter leurs contributions dans une Poster session l'indiquent sur la page de titre. Le Comité scientifique fait son choix en janvier 1984.

3.2 Choix des contributions

Conformément à une décision provisoire prise sur la base du résumé, les auteurs sont priés d'envoyer le texte complet de leurs contributions, préparées selon les directives de l'AIPC, au Comité scientifique avant le 10 avril 1984. Le Comité scientifique est responsable du choix des contributions. Les auteurs sont informés de la décision du comité le 10 mai 1984.

3.3 Publication des contributions

Le Rapport Final, publié après le Congrès, contient les exposés magistraux, les conférences, contributions et posters à toutes les séances du Congrès ainsi qu'un extrait de la discussion libre et les conclusions des rapporteurs généraux.

Le Rapport Final est envoyé aux participants du Congrès, après le Congrès.

EINLEITUNG

1. EINFUEHRUNG

Ziel dieser Einleitung ist, über Vorbereitung und Ablauf des Kongresses einige Informationen zu geben. Weitere Auskünfte, insbesondere mit Bezug auf Poster Sessions, technische Exkursionen, Filmprogramm und Ausstellung sowie weitere Hinweise allgemeiner Art finden sich in der Definitiven Einladung, die im Dezember 1983 herausgegeben wird.

Für den 12. Kongress der IVBH, der vom 3. - 7. September 1984 in Vancouver, BC stattfinden wird, wurden insgesamt 16 Themen ausgewählt. Der vorliegende Einführungsbericht umschreibt diese Themen näher und dient damit als Grundlage für die Vorbereitung von Beiträgen von Mitgliedern der IVBH und anderen an einer aktiven Teilnahme interessierten Fachleuten.

2. ABLAUF DES KONGRESSES

2.1 Tagesthemen und Hauptreferate

Jeder Tag des Kongresses beginnt mit zwei dem Tagesthema gewidmeten Hauptreferaten:

4\. Sept. A Der Entwurfsprozess

Prof. Ben Gerwick, San Francisco, CA, USA
Herr Jean Muller, Clichy, Frankreich

5\. Sept. B Management von Planung und Ausführung

Prof. Angelo Pozzi, Baden, Schweiz
Herr Hiroatsu Takahashi, Tokio, Japan

6\. Sept. C Konstruktiver Ingenieurbau in extremen Verhältnissen

Prof. Luis Estava, Mexico City, Mexico (provisorisch)
Herr Wladimir Saphonov, Moskau, UdSSR

7\. Sept. D Aufbruch zu neuen Grenzen im konstruktiven Ingenieurbau

Prof. Edmund Happold, Bath, England
Dr. T.Y. Lin, San Francisco, CA, USA

Die Hauptreferate finden von 9 bis 10 Uhr im Grossen Saal statt und werden simultan übersetzt. Während dieser Zeit sind keine parallelen Veranstaltungen vorgesehen.

2.2 Plenarsitzungen

Das jeweilige Tagesthema wird von 10.30 h bis 12.30 h in einer Plenarsitzung im Grossen Saal mit weiteren simultan übersetzten Beiträgen vertieft.

Die Plenarsitzungen für die Tagesthemen A und B werden dabei von eingeladenen Referenten bestritten. Im Anschluss an diese Vorträge ist eine freie Diskussion vorgesehen.
Zu den Tagesthemen C und D hingegen sind Beiträge aus dem Teilnehmerkreis erwünscht. Auch hier ist Zeit für eine freie Diskussion ausgespart.

2.3 Seminare

Gleichzeitig mit den Plenarsitzungen des Vormittags und nachmittags werden an Seminaren Referate gehalten, die ebenfalls aus eingereichten Beiträgen ausgewählt wurden. Auch hier ist Platz für eine freie Diskussion. Mit Ausnahme der Seminare IV, VIII und XII werden die Referate nicht simultan übersetzt.

Seminare werden zu folgenden Themen organisiert:

I	Arbeitssicherheit im Bauwesen
II	Computergestützter konstruktiver Ingenieurbau
III	Tragwerke für Verkehrsmittel auf Eigentrassee
IV	Wärmetechnisches Verhalten von Gebäuden
V	Fortschritte in Entwurf und Berechnung von Stahltragwerken
VI	Windeinwirkungen auf Tragwerke
VII	Fortschritte in der Ausführung von Stahltragwerken
VIII	Wirkung von Schnee und Eis auf Tragwerke
IX	Entwicklungen bei der Planung von Stahlbeton- und Spannbetonbauwerken
X	Fortschritte im Ingenieurholzbau
XI	Entwicklungen bei der Ausführung von Stahlbeton- und Spannbetonbauwerken
XII	Aktuelles Thema des Jahres 1984

Für alle Seminare sind Beiträge aus dem Teilnehmerkreis erwünscht. Eine Ausnahme bildet das Seminar XII, für welches das Thema zum Zeitpunkt des Drucks dieses Einführungsberichts (Juli 1983) noch nicht entschieden war, um 1984 wirklich aktuell sein zu können.

2.4 Poster Sessions

Die folgenden Themen werden auch in der Form von Poster Sessions diskutiert:

- Der Entwurfsprozess
- Arbeitssicherheit im Bauwesen
- Computergestützter konstruktiver Ingenieurbau
- Neuartige Bauwerke
- Tragwerke für Verkehrsmittel auf Eigentrassee
- Wärmetechnisches Verhalten von Gebäuden
- Management von Planung und Ausführung
- Konstruktiver Ingenieurbau in arktischen Regionen
- Konstruktiver Ingenieurbau in Erdbebengebieten

Poster Sessions sind auch an IVBH Kongressen nicht mehr unbekannt. Im Vergleich zu sogenannten Arbeitssitzungen gleicht eine Poster Session eher einer Ausstellung, an der an einem ganz bestimmten Thema interessierte kleine Personengruppen sich mit dem jeweiligen Autor vor einer Ausstellungswand fachlich unterhalten. Diese bewährte Art fachlicher Auseinandersetzung führt zu lebhaften persönlichen Diskussionen. Richtlinien für die Vorbereitung von Posters werden vom Sekretariat der IVBH vorbereitet.

Posters können für jedes der unter 2.4 erwähnten Themen unterbreitet werden. Die Posters werden in einem Raum während einer bestimmten Zeit (normalerweise ein Tag) themaweise laut Kongressprogramm aufgehängt. Die Autoren der Posters werden nach einem festgelegten Zeitplan für Diskussionen zur Verfügung stehen.

2.5 Kongressprogramm

Das Kongressprogramm, das den Teilnehmern zusammen mit weiteren Kongressunterlagen am Kongress ausgehändigt wird, enthält den Zeitplan für Hauptreferate und alle weiteren Beiträge sowie deren Zusammenfassungen. Die Teilnehmer können sich anhand dieser Unterlagen ihr ganz persönliches Programm zusammenstellen.

3. BEITRAEGE

3.1 Einladung zum Unterbreiten von Beiträgen

Ausser für die Tagesthemen A und B sind Beiträge von Teilnehmern zu allen Plenarsitzungen, Seminaren und Poster Sessions sehr willkommen.

Teilnehmer, die einen Beitrag einreichen wollen, werden gebeten, dem Sekretariat der IVBH vor dem 1. Dezember 1983 den Titel, das betreffende Thema sowie eine einseitige Zusammenfassung in einer der drei offiziellen Sprachen der Vereinigung einzusenden. Der Beitrag muss sich direkt auf den Einführungsbericht beziehen. Autoren, die vorziehen, ihren Beitrag in der Form eines Posters zu unterbreiten, werden gebeten, dies auf der Titelseite zu vermerken. Das Wissenschaftliche Komitee wird aufgrund dieser Zusammenfassungen eine vorläufige Auswahl im Januar 1984 treffen.

3.2 Auswahl der Beiträge

Die Autoren der in dieser Vorentscheidung ausgewählten Beiträge werden eingeladen, den nach den Richtlinien der IVBH vorbereiteten vollständigen Text ihres Beitrags vor dem 10. April 1984 dem Wissenschaftlichen Komitee einzureichen. Dieses ist für die endgültige Auswahl der Beiträge verantwortlich. Die Autoren werden vom Komitee über dessen Entscheid bis zum 10. Mai 1984 benachrichtigt.

3.3 Veröffentlichung der Beiträge

Der Schlussbericht wird nach dem Kongress herausgegeben und enthält zusätzlich zu den Hauptreferaten zu den Tagesthemen die Beiträge zu allen übrigen Kongressthemen einschliesslich Poster Sessions sowie einen Auszug aus der freien Diskussion und schliesslich die Schlussfolgerungen der Generalberichterstatter.

Der Schlussbericht wird allen Kongressteilnehmern nach dem Kongress zugestellt.

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Main Theme A

The Structural Design Process

Le processus du projet

Der Entwurfsprozess

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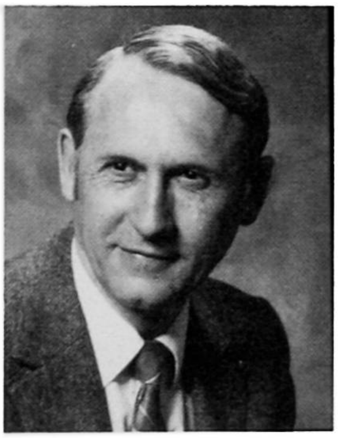

James G. MacGregor is chairman of the Canadian concrete code committee, a member of the structural design committee for the National Building Code of Canada and of the American Concrete Institute code committee. Dr. MacGregor has published extensively in the fields of shear strength, stability and safety of concrete structures and is a partner in MKM Engineering Consultants.

SUMMARY
This paper studies various aspects of the structural design process and suggests future trends in the fundamental bases of design. Topics discussed include limit states design, plasticity theory, finite element analyses, and the role of codes and computers. This paper is intended as the introduction to a session on such topics as problem identification and planning, choice of structural concepts, modelling and analysis, and feedback from erroneous design and construction.

RESUME
Divers aspects du processus du projet et les développements possibles des bases fondamentales du projet sont présentés dans cette contribution. Parmi les sujets considérés, il y a lieu de mentionner le calcul à la rupture, la théorie de la plasticité, l'analyse par éléments finis, ainsi que le rôle des normes et des ordinateurs. Cet article sert d'introduction à une séance générale sur des sujets tels que l'identification d'un problème et sa conception, le choix du système structural, l'étude et l'analyse par modèle et les leçons à tirer d'erreurs dans le projet et l'exécution.

ZUSAMMENFASSUNG
In diesem Beitrag werden verschiedene Aspekte des Entwurfsprozesses erörtert und zukünftige Entwicklungen in den Entwurfsgrundlagen aufgezeigt. Behandelte Themen sind unter anderem Bemessung auf Grenzzustände, Plastizitätstheorie, Berechnungen mit finiten Elementen, und die Entwicklung im Normenwesen und bei der Anwendung von Computern. Dieser Bericht soll als Einführung zum Thema dienen, und zu Beiträgen über weitere Aspekte wie z.B. Problemstellung und Planung, Wahl eines Tragwerkes, Modellberechnung sowie Lerneffekte aus Fehlern bei Entwurf und Ausführungen anregen.

1. WHAT IS STRUCTURAL ENGINEERING?

The Institution of Structural Engineers of England have defined structural engineering in the following manner [1]:

> "Structural engineering is the science and art of designing and making with economy and elegance, buildings, bridges, frameworks and similar structures so that they can safely resist the forces to which they may be subjected."

In a recent paper on the future of structural engineering Bobrowski [2] has presented this definition:

> "... structural design ought not to be limited to 'merely making forces change direction'. It must aim at optimum resolution of conflicts imposed by gravity, wind, earthquakes and temperature changes, as well as create an open space for whatever activity is required ..."

These definitions emphasize the interplay of science and art or craft, and emphasize that the structual engineer's end product must be buildable, useful and durable.

The engineering design process involves the following stages:

1. Problem identification and planning - A study of what is needed, why it is needed, the objectives and criteria to be used, and the resources available.

2. Generation of conceptual solutions to the problem - This is the crucial and creative stage of the design process. It builds strongly on the data collected in the first stage and on the knowledge, experience and intuition of the designer and his associates.

3. Appraisal of the consequences of the various conceptual solutions - The implications and consequences of each solution are evaluated to determine which solutions are practical, economical, and have the necessary aesthetic or other attributes.

4. Decision - The concept to be used is selected.

5. Checking, evaluation and elaboration - This stage involves:
   (a) definition of the loads and actions to be considered,
   (b) definition of limit states which could constitute failure,
   (c) structural analysis of the concept which has been chosen,
   (d) proportioning of members and fitting the members together into a buildable whole, and, finally
   (e) drawings and specifications must be prepared.

6. Construction

7. Testing and feedback - During construction the structure is monitored to determine whether it is being constructed in accordance with the design. After construction it should be monitored to determine whether its behavior corresponds to that assumed in the design. Maintenance problems constitute a further "test" of the design. Feedback from erroneous design and construction allows refinement of future designs.

The emphasis placed on each stage varies from engineering discipline to discipline and design to design. In the design of a major bridge, the structural designer and his team will carry out or be associated with all seven stages. In building design however, the architect and/or the client frequently assume responsibility for the definition of the problem and the selection of the major components of the conceptual solution, leaving the structural engineer involved only in the fifth stage - checking, evaluation and elaboration of a pre-conceived design. More often than not this leads to a less than optimum solution since the architect is working outside his area of expertise and the structural engineer is required to follow blindly.

In geotechnical engineering, particularly in the case of dams, slopes and deep excavations, the testing and monitoring of the structure during and after construction, and the feedback from this monitoring to the designer and the professional community is an essential stage in the design process. So much so, that a geotechnical engineer who did not monitor a major design would be considered in dereliction of his duties as a professional. In structural engineering, on the other hand, instrumentation and post construction monitoring are almost unheard of. Structural engineers believe implicitly that their analyses and the underlying assumptions are correct and nothing can go wrong. Post construction inspections do not generate revenue and, in addition, are discouraged by many insurers of structural designers.

Structural engineering, as it is taught at many universities and seen by many practitioners, centers around stages 5(c) (d) and (e) in the above list. These steps, particularly 5(c), structural analysis, are amenable to a pseudo-scientific treatment and decisiveness that appeals to many people who enter engineering. If the basic decisions can be made by a computer analysis, the engineer will not have to make them. The reasons for this emphasis on scientific treatment become more evident when we examine the personality characteristics of engineers.

2. CHARACTERISTICS OF ENGINEERS

Dr. Charles Goshen, a psychiatrist who has studied various occupations and professions, has reported a very high consistancy in the character traits of engineers as determined from standard psychological tests. He describes these as:

> "The engineer's most obvious characteristic is his precision, his meticulousness, his attention to detail and accuracy, or his perfectionism. Another striking quality is his intelligence ...[which]... tends to be used in a very specialized way. There is an obvious lack of broadness in point-of-view so that the superior intelligence he has is restricted to a narrow field with the result that he is likely to know a great deal about a little bit but knows only a little bit about the world at large.
>
> "He seems to exhibit an enormous need to be right. Actually, when we get to know him, we find he is primarily interested in trying to avoid being criticized for being wrong. As a result he demonstrates an outstanding sensitivity to criticism.
>
> "... The engineer is most useful to his organization when he has new ideas and develops them in collaboration with other new ideas. ... However the engineer's fear of taking a chance with a new idea, based on his dread of failure and criticism often prevents him from coming up with new ideas his organization needs."

These excerpts have been quoted because they give us insight into a major problem in structural engineering practice - the acceptance of new ideas and new methods. When the seven stages in design listed earlier are considered in light of Goshen's comments we see that the stereotype engineer should be particularly able to carry out stages 3 to 6, especially stage 5, but will tend to be less able or interested to deal with stages 1 and 2, and may be sufficiently concerned about the outcome of stage 7 to overlook it.

3. STRUCTURAL THEORY AND CALCULATIONS

Structural engineering has gone through stages and fads. Prior to the 18th century major structures were conceived by craftsmen who had a great knowledge of materials and construction methods, but little understanding of structural theory. During the 17th and 18th centuries the principles of statics were derived, the concept of elasticity arose, and such things as the theory of virtual work developed. In the latter half of the 18th century these concepts began to affect structual design. During this period Coulomb worked on the theory of earth pressures on walls and later on the strength of vaults. This work was based on "limit states" concepts in which Coulomb postulated modes of failure and used these to derive solutions.

The development of elastic stress analysis gradually, but effectively, displaced other methods of calculating the strength of structures and the concept of limit states design slipped into the background.

3.1 Limit States Design

Limit states design is a process in which the designer:

(a) determines all potential modes of failure. These are referred to as "limit states" since they represent states at which the structure has reached its limit of usefulness.

(b) determines relationships between loads and material properties and each of the limit states, and

(c) uses these relationships, etc. to select a structure which will not reach any of the limit states during its lifetime. This involves judiciously chosen margins against onset of failure, referred to as safety factors.

This is the traditional design process used by Coulomb, as mentioned earlier, and currently used in geotechnical engineering. It should be noted that this definition of limit states design said nothing about probability or partial factors of safety. These are merely "bells and whistles" added to the basic concept.

With the advent of elastic theory, structural engineers gradually drifted to the comfortable point of view that by satisfying the limit state of "no yield or fracture at working loads" they automatically satisfied all others. As a result, structural engineering practice gravitated to the simple, and generally safe, working stress design based on satisfaction of one, or at the most, two or three limit states.

In reinforced concrete, the earliest design procedures were related to observed modes of failure. Further "progress" led to the straight-line theory and further "progress" still, brought design to a pseudo-limit states design where all design was based on avoidance of ultimate limit states with "deemed to satisfy" checks of serviceability limit states.

In the application of limit states design, one limit state or family of limit states is judged to be more important than the others. This "primary limit state" serves as the basis of design. Once the initial choices of member sizes, etc. have been made on this basis, the remaining limit states are checked. Thus, in the design of bridges or buildings, current design codes assume that strength is the primary limit state and serviceability and durability are secondary. In foundation design the limit state of differential settlement will frequently be primary. The resulting designs are then checked for strength and other limit states.

Recently the British Standard BS5337 has proposed a design process for water-tanks in which water-tightness is the primary limit state, serving as the basis of design. Other limit states such as strength are then checked. Here, to quote Hardy Cross slightly out of context: "One might almost say that its strength is essential but otherwise unimportant."

In bridge deck design, resistance to corrosion is of equal importance to strength as the primary limit state. This is recognized in the 1979 Ontario

Highway Bridge Design Code, a pioneer limit states design code for bridges.

Limit states design has been widely condemned, particularly in England, as being too complex. This stems from the introduction of the CP110 code for concrete structures which simultaneously moved in three directions - it moved from a working stress format to a limit states format including load factors, it incorporated much more comprehensive proportioning rules, and it marked the change from Imperial to SI units. Any one of these changes would have brought complaints, to make all three simultaneously brought a storm of complaints. There was no similar groundswell in the U.S. and Canada when ultimate strength design of reinforced concrete was introduced in 1963 because the changes made were the bare minimum needed to implement the new design method. In Canada the limit states design code for steel structures is widely used and respected. Every effort was made in writing this code to avoid complex formulations. This must be the guiding principle for committees writing limit states codes.

The use of limit states design will spread to materials other than concrete in the next decade. This will be accompanied by an increased awareness and understanding of the design process as engineers formally consider the various limit states.

3.2 Safety Provisions

The reintroduction of limit states design into structural engineering practice occurred at the same time as probability based reliability studies were being developed. Modern safety provisions are based on extensive statistical and probabilistic work. In most cases these assume failure is due to variability in loads and member strengths since these are the terms the engineer deals with in his analysis and proportioning.

Increasingly, however, it is being recognized that the incidence of failures due to overloads or understrengths is small compared to that due to "gross errors" - human mistakes in the design office or at the job-site. Although it is fair to say that knowledge is expanding rapidly in this area, we currently do not have adequate procedures for avoiding this type of problem.

Rational treatment of gross errors will develop during the next decade leading to safer designs and fewer structural collapses.

3.3 Plasticity Theory

Material behavior can be idealized as consisting of an "elastic" domain and a "plastic" domain. For almost 200 years structural design has been based on elastic theory which assumes that structures display linear response throughout their loading history, ignoring the post-yielding stage of behavior.

The plasticity theory, on the other hand, disregards the elastic distribution of stresses, substituting an arbitrarily chosen distribution of forces which is in equilibrium with the loads and which corresponds to the kinematics of the structure and the material properties. The plasticity theories are not applicable at service loads but can be used to estimate the strength of concrete and steel structures.

In elastic theory, a given set of members, loads and assumptions leads to a "unique" set of internal moments and forces which may or may not approach the correct values, depending on how well the assumptions represent the real structure. In plasticity theory, it is up to the engineer to choose the desired set of forces and moments within certain constraints. When used as a design tool, there is no unique "correct" solution. If the structural engineer's personality is as characterized by Goshen, it is clear that few engineers will welcome the freedom and understanding gained from using plasticity methods when the "exact" and scientifically pleasing solution available from elastic analysis is considered to be above criticism.

Current design practice for reinforced concrete structures is a curious blend of elastic analysis to determine forces and moments, plus plasticity theory to proportion cross-sections for moment and axial loads, and empirical mumbo-jumbo to proportion for shear and torsion. The combination of elastic analysis and plasticity methods of dimensioning sections can be justified in most cases as a "lower-bound" plasticity solution to the problem.

One of the most important advances in reinforced concrete design in the next decade will be the extension of plasticity based design procedures to shear, torsion, bearing stresses and the design of discontinuities in structures (joints, corners, etc.). Equilibrium (lower-bound) methods of plastic design allow a designer to follow forces through a structure. Major steps have already been made in this direction.

3.4 Finite Element Analysis

The opposite of plasticity theory is elastic finite element analysis. Here by breaking the structure up into small elements it is possible to get an estimate of the internal stresses or strains. Again, the estimate is only as good as the assumptions which, for economic and practical reasons generally oversimplify the problem to cut down on the number of elements or to simplify element layout.

The finite element approach is basically incompatible with modern limit states methods of proportioning concrete structures. The analysis produces elastic stresses, the design methods are based on forces and moments.

In reinforced concrete design, finite element analyses are used when ordinary beam theory breaks down. In these problems, however, the stresses predicted by conventional elastic continuum, finite element analyses are almost meaningless since they neglect the redistribution of stresses due to cracking. It is essential that proper guidance be developed to aid engineers in making the transition from finite element analyses to selection of reinforcement. This must be done by persons who have a first-hand understanding of the response of unusual reinforced concrete members and not by elastic analysts. Professor Schlaich's recent work on this problem bears mention here [3].

3.5 Loads and Actions

The use of probilistically based procedures for deriving safety factors requires a knowledge of the variabilities of the resistances and of the loadings. This, and a number of dramatic failures, has led to attempts to define the true characteristics of wind, snow, live and other loadings. The resulting loadings have become more difficult to understand and more complex to use in design. Thus, the 1937 British Standard BSS 449:1937 covered design clauses, wind and other imposed loadings in the equivalent of 15, A4 size pages. By contrast the section on wind alone in the 1972 British Code of Practice was 49 pages. As another example, the 1980 Canadian National Building Code requires 12 different exterior loading cases when considering the effect of wind on a low, unsymmetrical building.

A major need today is more knowledge on loadings, coupled with attempts to simplify the description of these loadings in codes. Increased definition of imposed deformations is also required along with ways of coping with them in design and appropriate safety factors.

The structural design process copes poorly with interactive soil loadings as encountered on retaining walls, tunnel liners, etc. A part of the problem here is the inability of the geotechnical and structural engineer to communicate. The structural engineer wants a well defined load of well defined probability of occurrence so that he can use load factors and design for ultimate limit states. The geotechnical engineer feels this request is naive in view of all the uncertainties involved. Clarification of this situation is an area requiring significant and well thought out research.

3.6 Calculation Procedure Model

A calculation procedure model is the group of components involved in carrying out a particular structural calculation. It includes:

(a) the existing background of engineering knowledge, such as mechanics and understanding of material properties,

(b) the methods of application of this knowledge in structural analysis and member proportioning and the assumptions necessary in these methods.

(c) the rules of thumb, safety factors, allowable stresses, etc. needed in the proportioning.

When one aspect of the calculation procedure model is changed, other changes may be required also. Thus, if the stress calculations are changed from beam theory to finite element procedures it may be necessary to change the allowable stresses or the way in which these allowable stresses are applied in design. Similarly, when ultra-high strength concretes (70-80 MPa, 10000 to 12000 psi) are used for columns, it is necessary to change the compression stress block constants developed for normal strength concretes. Other changes may also be needed.

Unfortunately, loading descriptions are being developed by one body, analysis procedures by another and structural proportioning rules by still another. This leads to a lack of fit between the various parts of the calculation procedure model. A major role of code writing committees is to minimize this lack of fit.

Beeby and Taylor [4] have pictured the current calculation procedure model as:

> "... the tendency of some engineers to do as much calculation as they can in the design stage. This design approach, used uncritically, leads to the dangerous tendency of which many of us may be accused: if it is possible to carry out calculations for a particular aspect of behavior then we do, if no method currently exists then calculations for this aspect of design are deemed unnecessary.
>
> More simply, we put forward our aphorism for bad engineering practice: I can, therefore I must - I cannot, therefore I need not ..."

This arises from complexities obscuring the true mechanics, from a fear of being wrong and increasingly from a fear of litigation.

4. THE ROLE OF CODES

Codification of structural design has been carried farther than in any other branch of Civil Engineering. Thus, bridge design codes give detailed guidance to all aspects of member proportioning, less guidance on foundation design and little or no guidance about bridge pier scour and hydraulic design even though scour and erosion are the most common cause of bridge failures in some parts of the world. The degree of codification and the increasing complexity of the codes is causing concern among structural engineers. Beeby and Taylor [4] sum up recent trends in codes and design practice as follows:

> "Designers call for a clear statement on every subject so they can get on with their job without wasting their time on research; checking engineers constantly call for comprehensive rules which may clearly be seen to have been obeyed in all possible circumstances. It is nevertheless ironical that these same people often claim publicly that they yearn for the days when 'engineering judgement' was kind and a building could be designed on a few sheets of foolscap. ... despite protestations to the contrary from many engineers, there is a demand for our codes to be more and more comprehensive and, unless very careful attention is given to their implementation, more

complex."

The complexity of modern codes stems from the rapid growth in structural technology and materials and the concurrent reduction in the size of structures. Codes suffer from a lack of definition of their purpose, inadequate care by code committees, and inadequate study of the changes by users.

4.1 Purpose of Codes

The purpose of a code must be well defined before the code is written. Traditionally codes have been of two types: regulatory documents or "building codes" written to protect public health safety and welfare; and "codes of practice" written to aid engineers in the design and construction of safe, economical structures which perform in a satisfactory manner. While related, these purposes are quite different, a building code need not concern itself with serviceability limit states (deflections, vibrations, etc.) except when these endanger the public. A code of practice should deal with such items. Frequently a building code will recognize a particular code of practice as being "deemed to satisfy" its requirements concerning structural safety. When this occurs, the code of practice becomes a regulatory document even though portions are outside the scope of public safety.

Closely related to the purpose of a code is its audience. Should codes be written for the building inspector, the designer, the contractor, the researcher, university students? It would appear that the building code should be written for the building inspector and the designer in that order. On the other hand, the code of practice should be written first and foremost for the competent designer. Horne [5] discusses the degree of competence required:

> "... A Code need not - and should not - explain how such fundamental calculations [plastic collapse load, elastic frame analysis and elastic critical load] are to be made. ... A sound and up-to-date knowledge by all structural engineers of structural theory is the first essential if we are to have technically effective and progressive codes. ... The result [of the lack of such knowledge] has been the necessity to transform Codes into recipe books. This I believe to be the ultimate reason for the apparently complex and unsatisfactory nature of some of our structural Codes."

4.2 Bases of Codes

Three rules should be followed in formulating code clauses:

1. Wherever possible, code provisions should be based on mechanical models. Thus, for example, the American Concrete Institute Code and other modern codes clearly set out a physical model for flexure - a beam is a tension force and a compression force which form a couple. Strain compatibility and equilibrium are invoked. Similarly, this code requires that the total moments in a slab panel must equal the statical moment. On the other hand sections of this code dealing with shear and torsion offer little insight into the mechanical workings of beams subjected to shear and torsion. In general, those code sections dealing with mechanisms which are fully understood (flexure, short columns) are short and to the point. Sections dealing with poorly understood concepts are long, tedious and complex. In my opinion the most important value of plasticity theory will be to offer simple, equilibrium based models of shear and other actions.

2. If it is necessary to introduce empirical constants or simplifying assumptions, the end result should be as simple as possible. However, "simplification" should not be taken so far that the mechanical model is lost. The derivation of simple rules may take considerable effort on the code-writers' part. The rectangular compression block used in concrete design is a simple approximation compared to the Hognested stress block, the Jensen stress block,

the parabola-rectangle and others. This simplicity is based on extensive and thoughtful research, based in turn on a professional understanding of the degree of complexity which could be tolerated in a design office. In selecting empirical constants, the sensitivity of the problem to each variable should be examined with an eye to omitting those variables which have insignificant effect in relation to the general uncertanties involved.

3. When design shifts from one range to another (say from deep beam design to normal beam design) the appropriate design rules should meet at a common point unless there is a mechanical reason why they should not. In the American Concrete Institute code, there is a 100 percent jump in the efficiency of vertical stirrups when one passes the empirical code boundary dividing "deep beams" from normal beams. This is clearly unacceptable. Codification based on mechanical models avoids such problems in most cases.

Current codes suffer in part because these concepts have not been considered in the code writing process. This is due in part to a tendency for codes to adopt the formulations of a particular researcher rather than general principles. This comes in part from the presence of researchers on code committees (although such persons frequently are more able and willing to devote time to code work than others), and in large part because of the voluntary nature of code developmental work in many countries which limits the amount of time which code committees can spend rethinking complex expressions.

4.3 Code Format

As a reaction to the complexity of modern codes, different means of presentation have been suggested. These include: two-level codes (a master code and a simplified code), moving detailed rules to handbooks or data sheets, the use of performance codes rather than prescriptive codes, the preparation of omnibus codes covering all uses of a given material, and finally, rearrangement of material in the most usable fashion. The pros and cons of these various possibilities are discussed in an earlier paper [6].

The development of a two-tier code with a separate section or document presenting only those clauses required in the design of conventional buildings shows some promise as a means of simplifying the everyday design process while retaining the ability to handle a multitude of cases. In preparing such a code the "simplification" should not be carried to the extent that equations no longer appear to be related to the mechanics of the problem. To do so removes the engineer one step further from the design process. Two-tiered codes introduce potential legal problems, however. In the case of a lawsuit about a structure designed using the simple code, the designer's legal defense would have to be based on the lack of more stringent requirements in the complex code for the problem at hand rather than whether the building met the code definition of simple building. This, then, controls the scope of such a code and, in effect, requires that designers must be completely familiar with the complex code before using the simple code.

The use of the performance code format accompanied by a non-standardized recommended practice introduces uncertainties as to the relative acceptability of the various clauses in the recommended practice. Once a recommended practice was recognized by a building official, it would then become a mandatory document for all intents and purposes, thereby removing many of the perceived advantages of the performance code.

In recent years, the structure and arrangement of building standards has been studied and means are available for optimizing the organization of clauses. Rearrangement of code clauses according to such a scheme will not remove the technical complexity but may ease the task of moving from clause to clause.

The greatest simplifications will come, however, from clarification of the mechanical models to be followed in design and the use of these models as the framework for drafting of the code.

5. THE ROLE OF COMPUTERS

Computers have had, and will continue to have, a dramatic effect on structural design practice. Changes have occurred so rapidly that the profession has yet to assess and allow for the implications of these changes. In the calculation phase the computer reduces the drudgery of engineering computations and allows these to be much more extensive and "accurate". It assists in preparation of drawings and specifications and can be used to do material take-offs, etc. directly from the data files for these drawings. Specifications can be stored, amended and produced semi-automatically.

Because structural analysis and detailing programs are complex, the profession as a whole will use programs written by a few. These few will come from the ranks of the structural "analysts" referred to by Tedesko and Billington [7] and not from the structural "designers". Generally speaking, their design and construction site experience and background will tend to be limited. It is difficult to envision a mechanism for ensuring that the products of such a person will display the experience and intuition of a competent designer.

In the design office the reduction in computation time will free the engineer to spend more time in creative thought - OR it will it allow him to complete more work with less creative thought than today. Because the computer analysis is available it will be used. Because the answers are so precise there is a tendency to believe them implicitly. The increased volume of numerical work can become a substitute for assessing the true structural action of the building as a whole. Thus, the use of computers in design must be policed by knowledgeable and experienced designers who can rapidly evaluate the value of an answer and the practicality of a detail. More than ever before, the challenge to the profession and to educators is to develop designers who will be able to stand up to and reject or modify the results of a computer aided analysis and design.

Co-ordination problems arise in computer-aided design and drafting unless all consultants on a given project use the same software. If, for example, the architect updates his files and the mechanical or structural engineer's files remain as they were, some of the advantage of computer-aided-design is lost. In today's rapidly expanding computer market it seems unlikely that all consultants who might interface on various jobs will have the same hardware. Steps must be taken therefore to standardize the software so that interchangeability is possible.

To close this section, I wish to quote from Tedesko and Billington [7]:

> "We are fully convinced that better design depends upon the human interaction between analysts and designers. The computer certainly can help but it cannot replace the analyst who directs it. No one has been able to show that the computer has led to better structures. The computer played no role in the major new design ideas in concrete structures, such as Finsterwalder's segmental prestressed cantilever bridges, Isler's thin shell concrete roofs, and Khan's expression of concrete structure in tall buildings. It was the personality of the individual designer which led to the new forms."

6. ROLE OF RESEARCH AND DEVELOPMENT

About 0.35% of the turnover in the construction industry in England is spent on formal research and development, compared to about 36% spent on repair and maintenance [8]. Although this probably underestimates the fraction for research and development if one includes innovative design as "research", the percentage is still well below that spent in more innovative industries.

Frequently, developments in structural engineering are accepted in practice with little consideration of the long term effects. Examples have been the problems with High Alumina Cement, with certain types of glue-laminated timber beams, or more recently the increased incidence of corrosion failures of unbonded tendons in prestressed concrete structures. Reinforced concrete parking garages and bridge decks have proven highly susceptible to the de-icing chemicals used on roads. These members all behaved well in short time tests. It is difficult to visualize acceptance procedures which could adequately account for time effects - manufacturers will not sit around for 20 years waiting for approval, and frequently when long duration tests have shown one product to be better than another, the manufacturers have changed their processes during the interim so that the results are meaningless. One alternative would be a more intensive application of the spirit of limit states design by implicit consideration of the "limit state of long time chemical or physical stability" during the acceptance of new materials.

The results of research affect structural engineering practice primarily through inclusion in building codes and practitioners are, rightly so in most cases, hesitant about acceptance of research results without the screening of a code committee. As stated earlier, the code committee must very carefully screen new ideas, modifying them in light of past experience, simplifying them, generalizing them and making sure they are not in conflict with existing code clauses. A code does not exist to immortalize researchers and very seldom can a researcher stand far enough back to see all the implications of his work.

As stated at the beginning of this paper, the last stage in engineering design is to monitor the behavior of the completed structure. If properly carried out, this monitoring leads to ideas for future research and tempers the acceptance of new research.

7. SUMMARY

The next decade will be a period of great change in the structural engineering field. These changes will arise form expansion of our knowledge and introduction of computer aided design. The major changes will include:

1. Introduction of the true concept of limit states design - that design should itemize and consider all potential modes of failure.
2. Design methods based on plasticity theory will improve our design of concrete structures, particularly for shear, torsion, and localized loadings.
3. Widespread acceptance of computer aided design will reduce the drudgery, allow more time for proper review and improvement of designs.

At the same time, however, the profession will have to cope with problems arising from the rapid changes in our practice:

4. Means must be found to develop structural intuition in the increasingly inhibiting environment of computer aided design. This is a major challenge to engineering associations and educators but even more so to organizations engaged in structural design.
5. The incompatibility between finite element analyses and modern limit states design must be bridged, particularly in the case of reinforced concrete.
6. Codes should be based where possible on clearly explained mechanical models of structural processes. Code committees must work to express

research results in terms of generalized models. Code users must update themselves to the current technology.

7. Industry wide standards should be developed for computer-aided-design and drafting software to minimize problems of interchange of design information between architects and engineering disciplines.

8. Mechanisms must be developed to monitor structural behavior and to feed this information back into the codification and design process.

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Main Theme B

Engineering and Construction Management

Gestion du projet et de la construction

Management von Planung und Ausführung

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SUMMARY

By means of this Introductory Report, authors of building practice works are invited to relate practical transferable experience which they have gathered in the fields of Engineering and Construction Management. The author offers his contribution in presenting all essential elements of Construction Management and demonstrating their correlativeness. Differentiation is made between the systems and their actual application and supplementation.

RESUME

Ce rapport introductif est une invitation aux spécialistes des grands chantiers à faire part de leurs expériences relatives à la gestion du projet et de la construction. L'auteur présente les éléments essentiels de la gestion de la construction ainsi que leur interdépendance. Différents systèmes sont présentés, de même que leurs applications et développements.

ZUSAMMENFASSUNG

Mit diesem Einführungsbericht werden Autoren aus der internationalen Baupraxis eingeladen, aus ihren übertragbaren Erfahrungen auf dem Gebiete des Engineering und Construction Managements zu berichten. Als Hilfestellung führt der Author alle wesentlichen Teilgebiete des Construction Managements auf und bringt sie in einen Zusammenhang. Es wird zwischen den Systemen unterschieden und ihrer Fortschreibung und Handhabung.

0. ENGINEERING AND CONSTRUCTION MANAGEMENT

Management is comprising of all procedures which are necessary for the purpose of methodically planning and organising the execution of work activities and to bring the work to a satisfactory conclusion within the scheduled parameters. Therefore management is important both for engineering schemes as well as for executing a construction project. For this purpose management techniques based on "systems" [1] can be applied. Tender estimates, contract packages, or network programmes etc. are examples of a "system". They require to be continuously adapted to changing situations of which the execution of a construction activity is a typical example. The system has to penetrate the problem in detail without becoming too heavy handed nor lacking flexibility. The system also has to contain control standards whereby one may ascertain that the course being persued is the correct one. The system should also afford suggestions as to the corrections to be adopted in given situations. In all events individual systems require to be compatible with each other and must be integrateable. It follows therefore that management is the establishment of such systems and their continuous application and adaptation.

The design of a building may also be regarded upon as a system, but it is emphasized however that this is not a dynamic system. Once established the design will be adhered to in its original form. Management systems however are in most instances considered to be dynamic systems. They must be capable of adjusting themselves to continually changing conditions in order to attain the overall objective in spite of all disturbances which may arise.

As engineering and construction activities are partly dependent upon human intervention it is difficult to predict, within a dynamic system, the negative and positive impact of the human factor. Fortunately the laws of material do not apply to man. Nevertheless the design engineer and the practitioner on site are faced with the difficulty of assessing the human factor and making allowance of same within a methodical application of a system. For the design engineer the gap between the anticipated and the actual result is too wide, while for the site man it often appears that theoretical aspects play too great a role. Therefore it is the practitioner who is specifically called upon by this introductory report to make known his experience regarding construction management methods in Session B. Only a brief allusion should be made to the project itself because the main objective is that the participants of Session B should benefit from special experiences and methods of construction management. Management can be dealt with within the fields of both engineering and construction. The application within the organisation of a construction company is of particular interest. This introductory report therefore is designed to motivate construction practice authors and to afford them introductory guidance.

1. ENGINEERING MANAGEMENT

The design of a project comprises amongst others, major organisation problems and it is submitted that these may be easilier solutioned by applying management techniques. Within this chapter specific emphasis is made to the ever increasing problem of turn-key contracts. Figure 1 hereafter compares the standard construction contract to the turn-key contract. Although the turn-key contract offers advantages such as comprehensive responsibility and a gain of time, major problems may be encountered. For example the period in advance during the phase of planning, in respect to the corresponding construction design work, is very short. It is essential in order to avoid major problems during the construction phase that planned procedures be adhered to. This is particulary true in regard to approval procedures and to the time allocated to those participating in the planning phase. The client himself if he takes an active part in the planning phase must be able and willing to adapt himself to the required tempo.

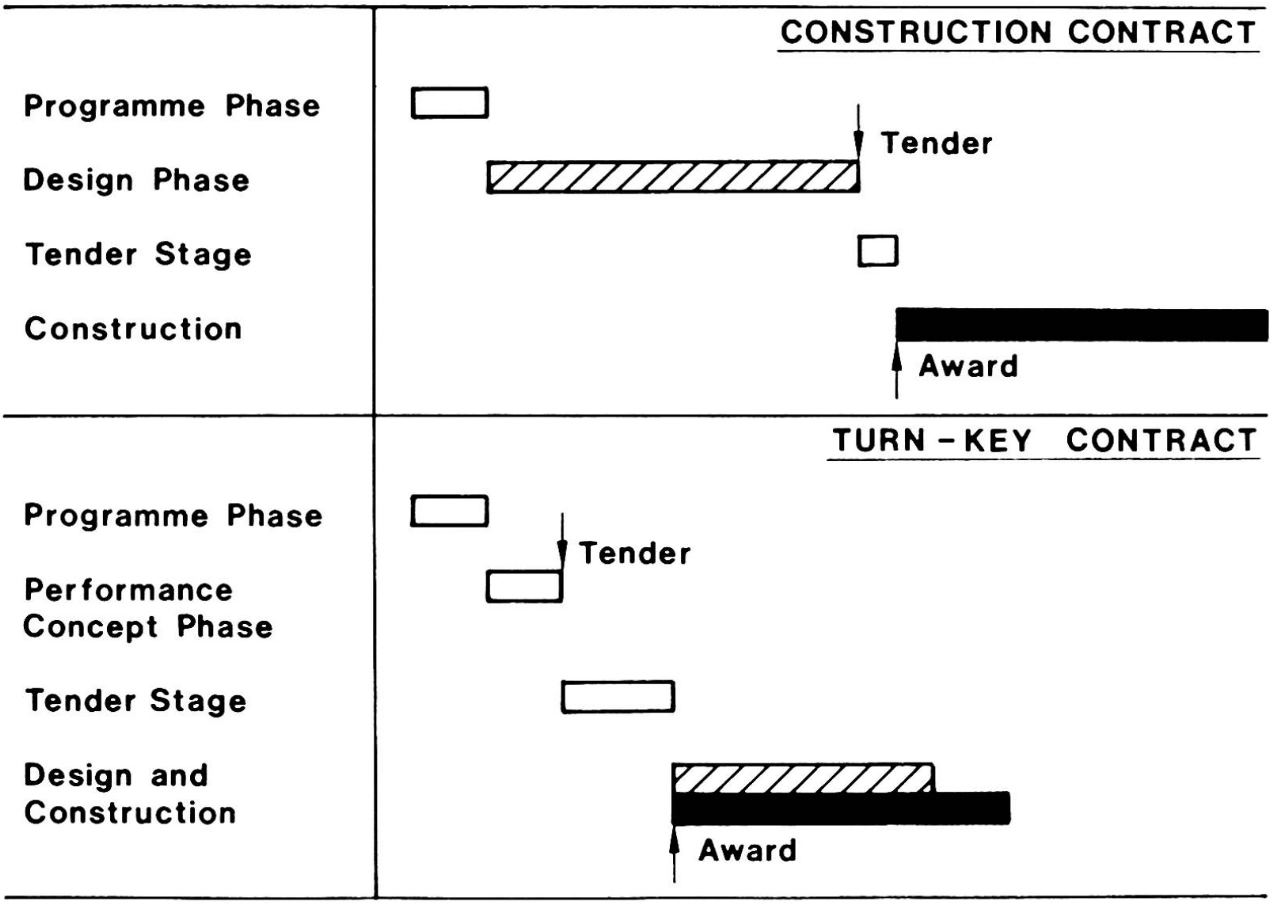

Fig. 1 Comparison of Construction and Turn-Key Contracts

2. CONSTRUCTION MANAGEMENT

More and more it is becoming obvious to what degree the costs of a construction project are influenced by the methods and procedures foreseen for executing the works. Due to the progress of technology we are compelled to direct our attention towards problems of site organization and process engineering. It is only since the latter years that these problems have been considered and investigated into in a methodical and scientific manner. The results gained from this research work will in the near future have a considerable impact on the design, the selection of construction procedures, the drafting of construction contracts and the cooperation of all participants in the construction market.

In view of this rapid developing phase of construction management it will be very interesting to present to a large public forum like the 12th Congress of IABSE in Vancouver the experience gained through the practical application of different methods for specific elements of construction management.

The following Fig. 2 illustrates the structure of the Management Organization System

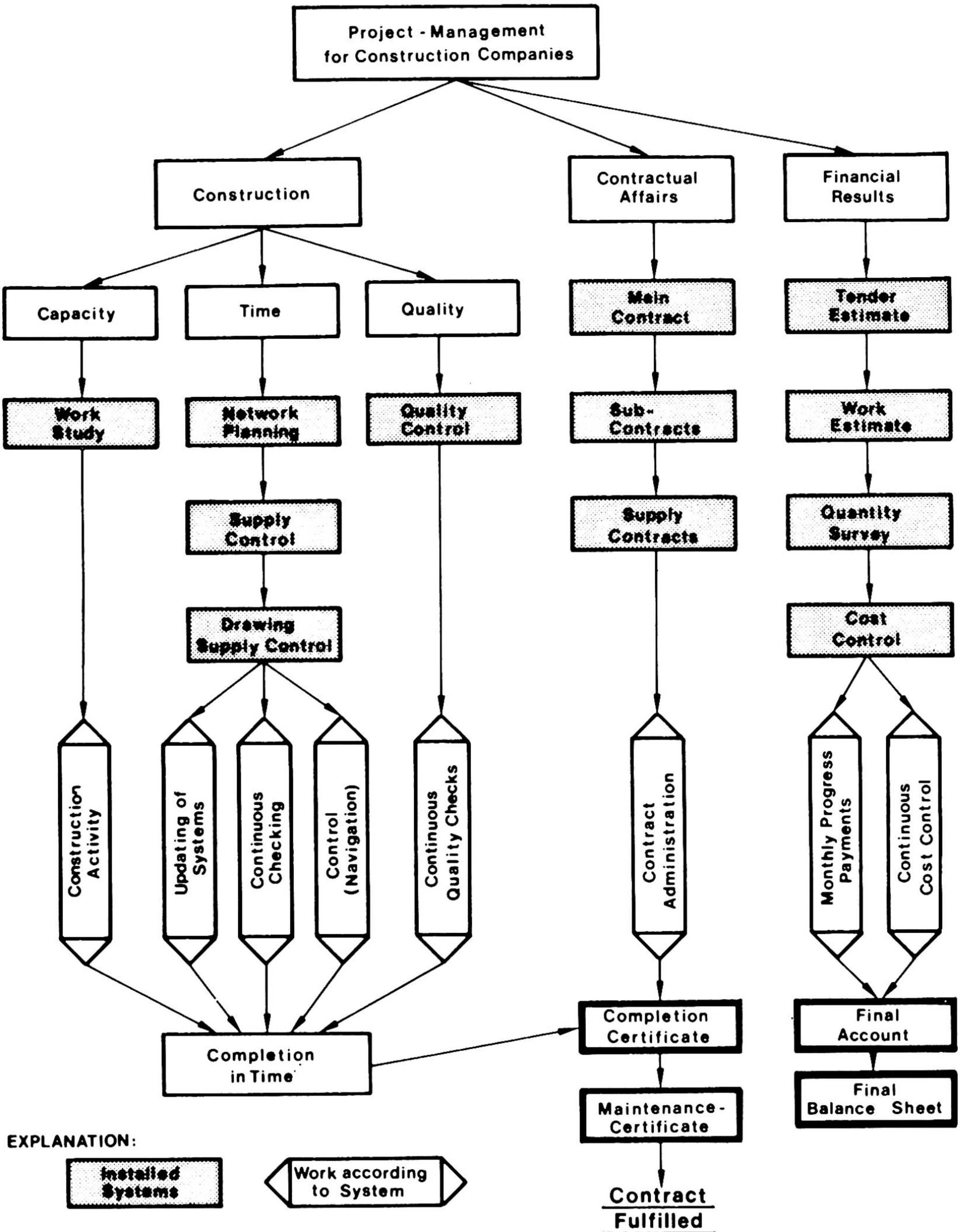

Fig. 2 Management Organization System

Fig. 2 shows the various elements of Construction Management
- Construction
- Contractual Affairs
- Financial Results

From each main element is originating one or several systems. Within each system (shaded box areas) the work to be executed for certain projects is structured, comprehended, distributed according to procedures and quantified. The vertical boxes coming next display the various steps and measures, which are to be taken during the construction phase, to ensure that the project will be completed in due time, according to the prescribed quality, and within an acceptable cost frame. In Session B we shall learn of the experience gained in practice with the various systems together with their application and continuous supplementation. The reciprocal action between the individual systems is also of importance. The interdependency is to be demonstrated between a partial performance, the required resources, the time allocated, the required quality, the provisions and obligations stipulated in the construction contract together with the cost of this part of the performance.

It is submitted that a workable system has to be flexible. The possibility should be available to parry both internal and external disturbances before they cause a domino effect to the whole construction site.

2.1 Capacity

To all partial tasks, resources are required to be allocated. These resources consist of various combinations of equipment and work forces. The efficiency of these performance groups is influenced by the human factor either positively or negatively.

2.1.1 Work Scheduling

In the system of work scheduling all the operative steps and their corresponding capacities are established. This implies that all construction work is theoretically anticipated. From the work scheduling is developed the strategic concept of the construction execution. This enables the establishment of equipment lists, the allocation of work forces, capacity calculations and the design of the site installations. The quality and thoroughness of the work scheduling is often decisive for the financial result towards the end of the construction work.

2.1.2 Construction Activity

The resources defined within the work scheduling are mobilized following the tactical requirements. The construction work is undertaken and the execution must be continuously rescheduled in order to make allowance for both internal and external disturbances to the planned execution. These disturbances have to be considered and adaptations made in conformity with other requirements of the Construction Management system. This is to ensure that the works are completed as planned, at the anticipated cost level, not withstanding the disturbances.

2.2 Execution in Time

Major construction projects often include long-term activities far off from the site. These require to be synchronized in such a manner that construction processes will not be hampered by difficulties with procurement and supply. If disturbances with supplies arise they must be neutralized immediately.

2.2.1 Network Planning

For process planning of construction activities a number of graphic and computer aided devices are available which allow the individual activities to be interconnected. It is a matter of controversy whether such systems, such as network planning, should combine large numbers of activities in a close woven network or whether preference should be given to a less denser network which offers a wider range of possibilities, when considering the daily routine of construction processing. The presentation of such a network is also a matter of differing opinions. Practitioners who are concerned with construction supervision advocate for networks based on visual arrows which are simple to comprehend.

Construction firms however generally prefer to represent the individual activities as a node. The node serves as a description depicting all characteristic elements of an activity such as time, equipment, work force and budget. Another factor in favour of the node network is its adaptability to computer techniques. It would be of interest to discuss these matters in Session B.

2.2.2 Supply Control

Disturbances to logistics such as the supply to the site of material, equipment and spare parts can lead to an economical disaster. It is not sufficient only to rely upon the contractual obligations of subcontractors and suppliers. The general contractor should develop a specific supply system connected to his general network planning. Thus deliveries, acceptances and timely completion may be scheduled, controlled and navigated in a satisfactory manner within the site network planning. Past experience has shown that a supply control based on the site network planning can considerably improve the monetary result of the site.

2.2.3 Drawing Supply Control

The late delivery of drawings is one of the most fatal events which a construction site may experience. Decisions may be made too late and drawings will not be compared with each other with regard to unmasking potential discrepancies. Approvals would also be given with delay. Therefore it is of paramount importance that a drawing supply control system is established which incorporates all disciplines participating in the project. The control system should be specifically provided for within the construction contract. The system should be conceived to allow timely and unbureaucratic controls, warnings and also the premature acceleration of particular matters.

2.2.4 Updating the Systems

The site network planning, supply control and drawing supply control require to be updated at reasonably brief intervals during the complete construction period and adaptations made following the progress of work. The scheduled completion date however should not be affected by these updating measures. The float times contained in the three networks are to be utilized as a means of minimizing the costs of those participating in the project. These three systems may also give warnings of eventual delayed performance of third parties. Quite often the client's or the engineer's shortfallings are the cause of disturbances. When this is the case the control systems assist in upholding the validity of a possible claim. Clients and courts of arbitration often require such detailed proof of causes and effects.

The primary function however of the three systems is to allow the scheduled activities such as supplies, performances and measurements to be adapted to the prevailing conditions of execution. Acceleration measures may also be adopted via contract administration while they are still possible or prior to them occasioning major costs. Directing and controlling the various intermediate steps and phases is also known as "navigation" in view of its similarity to the organization of work on bord a ship between the captain on his bridge responsible for bringing the ship to port and his navigator assisting him with all his electronic devices. A modern construction site requires also to be correctly navigated by means of early warning systems and corrections to the planned course. It would be too much to expect a construction manager to do all that is necessary on his own.

2.3 Quality

Exemplary construction performances may sometimes result in providing an inferior quality. It is becoming more and more a controversial question, especially where turn-key projects are concerned, as to which standard of quality is actually required. It is noteworthy to underline that the construction cost of a turn-key project has in certain cases practically doubled. Therefore within the system of quality control one has to differenciate between quality assurance and quality control.

2.3.1 Quality Assurance

Within the system of quality assurance a uniform quality framework containing no contradictions is established. The nesessary standards require to be determined.

2.3.2 Quality Control

Within the system of quality control the appropriate tests together with their frequence and statistical evaluation require to be ascertained. Reflexive control by the general contractor is becoming more and more the rule in lieu of the exception.

2.4 Contractual Affairs

A popular slogan intimates that the art of construction is the interpretation of the contract. In no other industry are individual contracts especially drawn up for the purpose of governing a specific project. It is to be understood that all the various conditions and descriptions provided for by the contract have meanings of considerable monetary importance. A bill of quantities containing contradictions or shortfallings may result to extensive expensive contention. A general contractor performs within a system comprising of the construction contract, the various subcontracts and the supply contracts. This system should be watertight, but in practice however this is not always the case. In the event of contradictions and shortfallings they invariably have to be supported by the general contractor.

2.4.1 Main Contract

Opinions differ as to whether a main contract should be drafted in detail or left relatively open. A further subject for discussion is concerning whether bills of quantities should be subdivided into numerous individual items or whether they should be drawn up in a summarized form. Concerning turn-key contracts the extent of the contractor's obligations arising from the design must be determined. An unequivocal price may at a late stage of execution be conflicting with the other parties' conceptions as to the suitability or extent of the required undertakings and their corresponding quality. Turn-key contracts are especially favoured by various developing countries. If a deficient performance concept is experienced in such a contract this will result in a major controversy.

2.4.2 Subcontracts

Subcontractors, especially those who provide sophisticated installations, are generally located in industrialized countries. They are required to fulfill the obligations of the main contract without deficiencies or shortfallings but quiet often this is not achieved. Furthermore practitioners do not always realise nor understand the difference between "subcontractors to be approved" and "nominated subcontractors". In the case of nominated subcontractors these are decided upon and selected by the client often with the help of his consultants. The client also decides upon the performance to be fulfilled together with the price and any other relevant matter. This nominated subcontractor however remains responsible towards the main contractor. In the case of "subcontractor to be approved", the selection, the performance and the price together with any other matter remain the full responsibility of the contractor without any restrictions. The client retains however the right and the possibility to exclude a proposed subcontractor.

2.4.3 Supply Contracts

The initial impression and understanding of certain main contractors is that in the case of supplies by third parties they will remain free from eventual problems. In practice however this is most certainly not the case. Other than the suitability of the supply itself problems may arise concerning who is responsible for the various modes of transportation. Also upon delivery on site who is responsible for defective or damaged goods. It is suggested that it might be preferable to abandon the monitary advantage resulting from separating delivery and performance and to favour a supply, ship and install contract providing fully comprehensive responsibility up to the final acceptance of the installation.

2.4.4 Contract Administration

In a general manner construction sites produce mountains of correspondence. Meetings are held and minutes established together with memorandums, telexes and incoming and outgoing letters. The engineer gives written or verbal instructions which are often subsequently modified by his representative. The contract drawings contain ambiguities and discrepancies and the contract documents as such often contain the roots of possible contention. A general ignorance and misunderstanding in regard to the duties and obligations agreed upon prepare the way to possible dissension. The remedy which enables one to overcome these difficulties, it is suggested, is "contract administration".

Contract administration properly established and systematically applied allows the contractor to control and to collate all the technical and contractual correspondence received from or to be sent to other parties. It is underlined that contract administration is isolated from the strain and responsibility of the site staff currently occupied with the execution of the works. As construction is synonymous to contract interpretation, the required link with contract administration is obvious. Furthermore the required standards of the work schedule together with network planning and quality control must also be enforceable via this system.

2.4.5 Completion Certificate

Practitioners are often faced with the problem of obtaining acceptance of the works at the scheduled date. "Substantial completion" has unfortunately a variable definition following that interpretation is made by the client or by the contractor. Past experience proves that the exact definition of the words, substantial or non-substantial is never envisaged at a very early stage of the project. Which functions and tests for example are a prerequisit for the obtention of a certificate of substantial completion. Is the client allowed to maintain that the acceptance tests may only be carried out at the prescribed dates or may the contractor insist that a completion certificate be established before the contract period has elapsed. Would a premature completion imply that the client has to provide his contractual contribution in advance of the scheduled provisions. If the scheduled provisions may not be provided prematurely then the benefit anticipated by the contractor through accelerating his works in order to obtain early completion will not be secured. It is submitted that difficulties with and eventual refusals of the completion certificate fall within the scope of a contractor's risk.

2.4.6 Maintenance Certificate

The FIDIC Conditions of Contract provide that the construction contract is concluded only when a maintenance certificate is delivered. In Germany and other European countries the provisions are different in the sense that the period of guarantee during which the remedial work is required to be carried out is separated from the actual construction contract period. In other words the period of guarantee is subsequent to the construction contract.

2.5 Financial Results

A contractor, it is submitted, undertakes construction works in the hopes of obtaining a profit. For the purpose of securing this profit a number of methods are available to him. Therefore the contractor must be judicious in his choice of the appropriate and adequate method required in each particular case.

2.5.1 Tender Estimate

A tender estimate may be established following different methods. These methods consist principally in either a simple estimation of prices, or an extrapolation of previous results, or by a detailed study of the project under consideration. If the proposed tender includes for an itemized bill of quantities then the estimate will in general be built up following the items provided in the B.O.Q. It is underlined that the total costs of the contractor must be covered or included within these items. The laws of probability indicate that numerous bids have to be made before a contract is awarded. Therefore the amount of time that one may afford to this matter must be parsimoniously employed notwithstanding the required exactness of a tender estimate. Various systems are available however which allow processing estimates by the means of electronic appliances. The utilization of a computer has the advantage of allowing performance values to be stored and thereby readily available for future reference.

2.5.2 Work Estimate

Upon a contract being awarded a fully detailed work estimate should be produced to enable the contractor to ascertain the extent of his commitments and to afford him a valid basis for the purpose of planning and executing the project. This requirement is not always fully appreciated by contractors. The modifications to the original estimate due to negotiations prior to the award also have to be introduced into the work estimate. The work estimate should also contain the necessary details to permit all preparatory works and site installations to be set up within planned parameters. Budgets may also be established for various sections or units of the works thus forming independant cost plus profit islands. In certain instances an internal separate bill of quantities may be established based on the requirements of the contractor's network planning and cost control requirements rather than on the itemization of the performances as per the contract.

2.5.3 Quantity Survey

In the event of a unit price contract, the actual work executed is calculated by way of measurement of work executed, either physically on site or from drawings, based on the items contained in the B.O.Q. This method is adopted both for the preparation of monthly progress payments together with the appraisal of the final certificate. Certain construction contracts provide for this activity to be entrusted to an independant Quantity Surveyor. In other instances the contractor remains responsible for the fulfillment of this requirement. The Institute of Quantity Surveyors whose members are specifically qualified for the purpse of carrying out the aforementioned activities has rarely penetrated the continental European construction market.

2.5.4 Cost Control

Cost control, it is submitted, is required to be initiated right from the offset of the construction process. The system should be such that a regular course of procedure is achieved thus allowing the results obtained to serve as a reference for future estimates as well as providing the necessary information for the project under execution. For this purpose various computer aided systms are available. This particular aspect might be found to be an interesting subject for the 12th IABSE Congress in Vancouver.

2.5.5 Monthly Progress Payments

Where construction contracts provide for monthly progress payments the normal procedure requires these payments to be reconciled with the engineer and the employer. Quite often opinions differ as to the degree of exactness required for these invoices. From a legal point of view they are merely invoices for partial payment. Particular attention should be given by all the parties concerned in minimizing as fas as possible the period of time which elapses between establishing the monthly account and the ultimate receipt of the funds in the contractor's account. Prolongation of this period of time always results in the contractor incurring a financial loss which in some form or other is borne by the national economy and therefore also by the client. It is submitted that clients should avoid providing interim financing via delayed monthly progress payments.

2.5.6 Continuous Cost Control

A continuous and systematic cost control is to be implemented during the complete construction period. This continuous cost control consists of comparing the scheduled and actual amount of work executed in a given time together with evaluating the anticipated final monitary result. Various systems for the purpose of achieving this aim are available. In one particular system, performances and/or costs are strictly limited to each target date, thereby avoiding that advance performances do not falsify the result. In other systems the works yet to be executed are pre-estimated and extrapolation is made on the basis of data deriving from accountancy records. This allows elimination of all problems regarding the delimitation of costs and performances. Attention however is drawn to the fact that a source of error is possible when a subsequent activity has been newly estimated because the anticipated values might be erroneous.

2.5.7 Final Account

In theory, and in a general manner in practice, the acceptance of the final account brings to an end the construction contract. After acceptance of the final account no other payments for any contractual performance may be entertained. The form and presentation of the final account is to be agreed upon in due time with particular thought being given to the measurement sheets and subsequent calculations.

2.5.8 Final Balance Sheet

In instances whereby the final account brings the construction contract to an end, as far as its external relationship in concerned (excepting outstanding claims), the final balance sheet marks the end of the contract with regard to its internal relationship with the construction company. The final balance sheet defines to what degree the contractor's anticipated profit has been secured. For major construction sites it may appear on paper that a profit has been achieved. This profit however might be smaller than the residual value of the equipment that has not been depreciated. In this particular case only the demobilization of the site and the ensuing sales returns gained from selling equipment, camps, spare parts etc., will decide if the profit margin as shown on the balance sheet is factual or not.

3. INFORMATION SYSTEMS

Notwithstanding the presentation of Figure 2 it is underlined that the 5 main chains and their subsystems are not independant of each other. To the contrary there is a marked interdependence between the various chains. It is to be understood that this interdependence is not pictorially displayed in Figure 2 to avoid confusing the reader. It would be interesting to learn from practical experience which integrated information systems are available to illustrate this horizontal flow-chart simply but adequately. A suitable computer coding system could perhaps be helpful in this direction. It is emphasized that various theoretical models pertaining to available information systems are too complicated to allow their application within normal site conditions.

4. RESEARCH FOR SOURCES OF LOSS

In the comprehensive method of construction management of which it is question in this paper it is implied that all essential production and controlling activities are "anticipated" following a plan and that these activities proceed according to these plans. In the event that an anticipated result fails to be achieved corrections are implemented during the subsequent navigation. A further procedure which allows the improvement of results and which offers assistance in counteracting disturbances is known as the "control circuit scheme". This procedure however is not for ascertaining if the scheduled target has been attained but is for measuring any eventual loss of productivity. It is perhaps regrettable that this procedure has not been given appropriate consideration within the construction practice. It would be of great interest to obtain information of practical experiences within this field and to have the opportunity of debating same.

Figure 3 hereafter displays the aforementioned control circuit scheme.

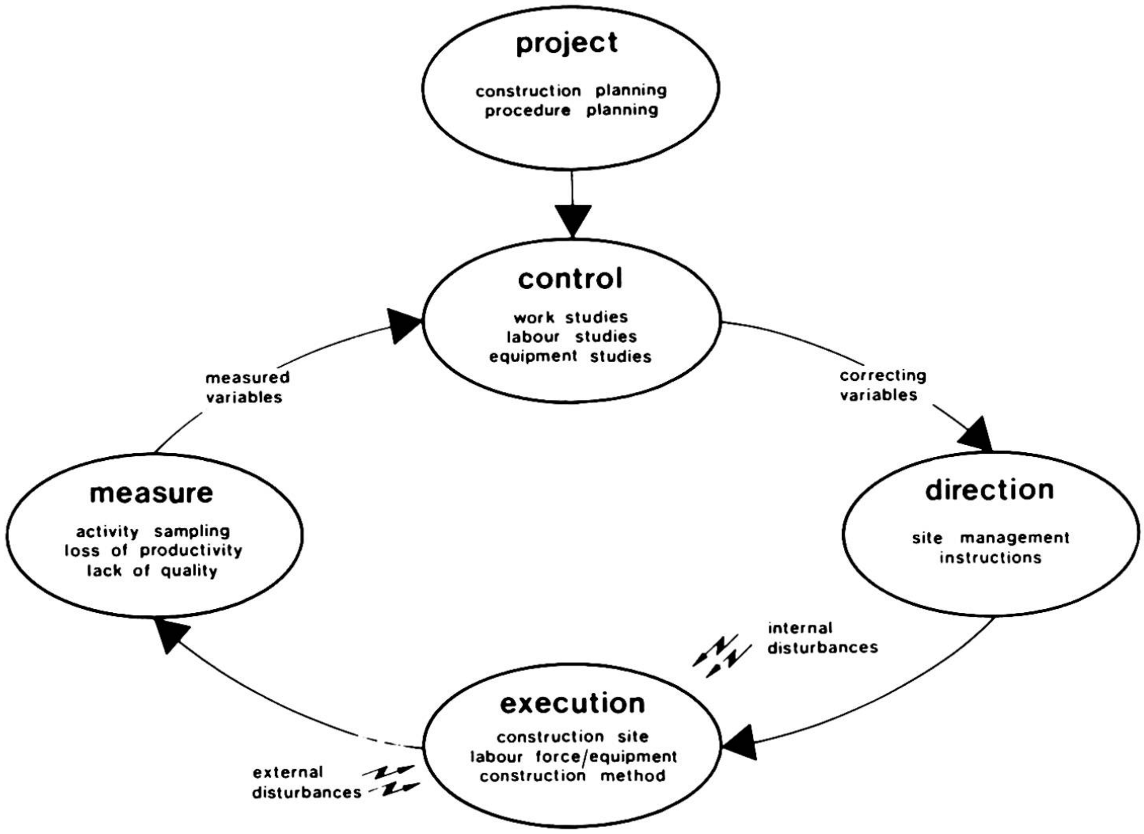

Fig. 3 Control Circuit Scheme

By means of methodical and systematic "activity sampling", observation of the current production activity of the labour force and equipment may be accomplished and statistically evaluated. [2] The "idle time" as measured is allocated to either internal or external causes of disturbance.
After affording the necessary corrections and adjustments to the workstudy the rectified information is recycled via the "control circuit" to those participating in the project. It would be of major interest to learn of experiences gained in working with such a "control circuit scheme" in actual practice and to have the possibility to debate its merits.

5. RISK MANAGEMENT

A systematic analysis of possible risk conditions is imperative in modern business. Risk has to be assumed otherwise no progress is possible. Risk may be defined as an exposure to mischance, the acuteness of which may be evaluated according to the probability of its occurrence and to the ensuing severeness of its consequences. It is submitted therefore that any risk inherent to a planned construction activity requires to be systematically ascertained. Risks require to be fully identified and evaluated in terms which will allow the contractor to accept or to refuse to cover the risk. [3] In the event that a particular risk is found to be unacceptable various alternative solutions are possible. Either the order or award is rejected, or the risk is eliminated or diminished by means of special measures of organization or by the selection of a more adequate construction procedure. Further possible alternatives are to transfer the risk to the client or to the subcontractors or to cover the risk, providing this is possible, by means of an appropriate insurance.

6. CONCLUSION

Experienced practitioners of Construction Management are invited to relate their particular experiences gained in the construction practice. A brief description of the project is required to illustrate the background of the subject matter under consideration. The members of the audience hopefully will be able to gather information and encouragement which will offer them help with their own particular projects and which also will allow them to draw their own conclusions. This Introductory Report is intended to be utilized by those participating in the Congress as an intermedium allowing relevant information and experience in the field of Construction Management to be either passed on, received or exchanged.

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Main Theme C1

Structural Engineering in Earthquake Zones

Structures de génie civil en zones sismiques

Konstruktiver Ingenieurbau in Erdbebengebieten

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T.P. Tassios, full Professor of reinforced concrete structures at the N.T.U., Athens, is the author of 120 papers and research reports on the theory of elasticity, concrete technology, seismic behaviour of R.C., etc., in several languages. Prof. Tassios is member of the Adm. Council of CEB and chairman of some international committees.

SUMMARY
This report is a short presentation of the practical design process for buildings in earthquake zones. It is also a call for papers dealing with problems of a seismic design, mainly the dimensioning of structures in reinforced concrete, as well as but to a lesser extent, in masonry and in steel.

RESUME
Le rapport expose le processus du projet de bâtiments situés dans des zones sismiques. Il tient lieu aussi d'appel aux communications relatives au projet, et plus particulièrement au dimensionnement de constructions en béton armé ainsi qu'en maçonnerie et en acier.

ZUSAMMENFASSUNG
Es wird kurz über den praktischen Entwurf von Bauten in Erdbebengebieten eingeführt. Der Bericht soll Beiträge hervorrufen, welche mit erdbebensicherem Entwurf zu tun haben, hauptsächlich bei der Bemessung von Stahlbetonbauten sowie Backstein- und Stahlbauten.

1. PREAMBLE

This introductory report is an attempt to make clear what kind of papers is wanted for this Session of the Congress. The Scientific Committee of the Congress has deemed appropriate to exclude papers dealing with:

- Repair, strengthening and redesign of structures damaged by earthquakes; the IABSE Venice Symposium (1983) was a better occasion.
- Description of recent earthquakes, (unless specific features of structural configuration or detailing were intended to be shown as systematically proved to be clearly advantageous or disadvantageous).
- Structural analysis under seismic conditions; the recent IABSE Structural Engineering Document entitled "Dynamic Response of R.C. Buildings" may serve several practical purposes in this respect.

On the other hand, the Scientific Committee has wished that the papers should rather concentrate on practical design problems (mainly dimensioning) under seismic conditions.

In an attempt to serve this purpose, this report starts by reminding the entire process of aseismic design. In doing so, a list of Session-topics will be built-up, except of those previously excluded. Parallelly, restating some old problems and raising some new ones is facilitated. Finally, papers are invited to offer answers to some of these design problems specifically.

2. THE PROCESS OF PRACTICAL ASEISMIC DESIGN

The following main steps may be distinguished in designing structures in earthquake zones.

2.1. Selection of the kind of materials and the structural system

After deciding the appropriate s i t e for the construction of the building, the main structural m a t e r i a l (timber, steel, masonry, reinforced concrete) is chosen and the structural system is generally decided upon: e.g., frame system, wall system, dual system.

2.2. Conceptual design, a very important step in the overall design: Structural c o n f i g u r a t i o n and empirically selected arrangement and rough dimensioning of building elements are carried-out, subject to analytical verification.

2.3. Assessment of the seismic conditions, usually expressed by a seismicity estimator, e.g. an effective peak ground acceleration max a_g and an assessment of local soil conditions (selection of a site coefficient "S").

2.4. Estimation of the natural period of vibration of the building as it has been conceptually designed. Only rough estimates are needed, in order to read normalised response accelerations ("α_T") out of a given design response-spectrum.

2.5. Evaluation of a base-shear-coefficint as specified by Codes. To mention the example of CEB Seismic Annex (1982), this coefficient may be expressed as

$$c = \frac{\gamma_n \alpha_T}{K} . S . \max a_g$$

a_g = effective peak ground acceleration

α_T = normalised response acceleration, a function of the natural period of vibration of the building; α_T is read-out of a locally valid design spectrum (usually given by Codes).

γ_n = partial safety factor, modifying the target failure probability as a function of the i m p o r t a n c e of the building,

K = behaviour factor, an overall empirical modification of the elastic to elastoplastic model (accounting for the d u c t i l i t y of the structure).

S = site coefficient (soil type), expressing the higher vulnerability soft soils impart to flexible buildings.

2.6. Selection of the structural model: The structural analysis under seismic actions is to be made by means of a more or less simplified model simulating the dynamic behaviour of the actual building (linear elastic models are used in the majority of normal buildings).

2.7. Design load combination, which takes into account the reduced probability of variable actions to be simultaneously present with the design earthquake actions.

2.8. Structural analysis (modal analysis, equivalent static analysis).

2.9. Check for the compliance of Design Requirements. In this respect, it is important to underline the significance of performance-oriented modern Codes, which make clear to the designer **requirements** and **criteria** to meet these requirements.

a) Safety requirement, fulfilled by the following means:

- Stability of the building as a rigid body, and foundations' stability.
- Control of failure mechanisms; normaly, it suffices to ensure that plastic hinges will first appear to beams than to columns ("capacity design").
- Check of the ultimate capacity of critical regions of all building elements, versus the action-effects found by the structural analysis.
- Care for appropriate qualities of materials and, mainly, for appropriate detailing, in order to ensure sufficient d u c t i l i t y - a fundamental property of an earthquake resistant structure.
- Quality assurance plans, being extremely more needed in earthquake situations than in any other case.

b) Limit deformations - a requirement aiming at the limitation of d a m a g e s and misfunctioning of the secondary (and most expensive) organism of the building.

3. IDENTIFICATION OF PRACTICAL PROBLEMS and CALL FOR PAPERS

Following the guide-lines set forth by the Scientific Committee of this Congress, the Session this Introductory Report is intended to serve, should focus on p r a c t i c a l problems in designing structures under seismic conditions. Among these problems conception and dimensioning seem to have a preference, although some everyday problems of structural analysis would also be considered as "practical" as well.

Under this optics of priority, this part of the Report is an attempt to reiterate the importance of some problems and to invite research workers and designers to offer their knowledge and experience contributing to the improvement of the solutions already given to these problems. Of course, the selection of these problems is to a certain degree arbitrary and it cannot be restrictive at all; however, if a collective effort is concentrated to certain problems only, some sound conclusions would be drawn out of this Session of the Congress, to the benefit of the profession at large.

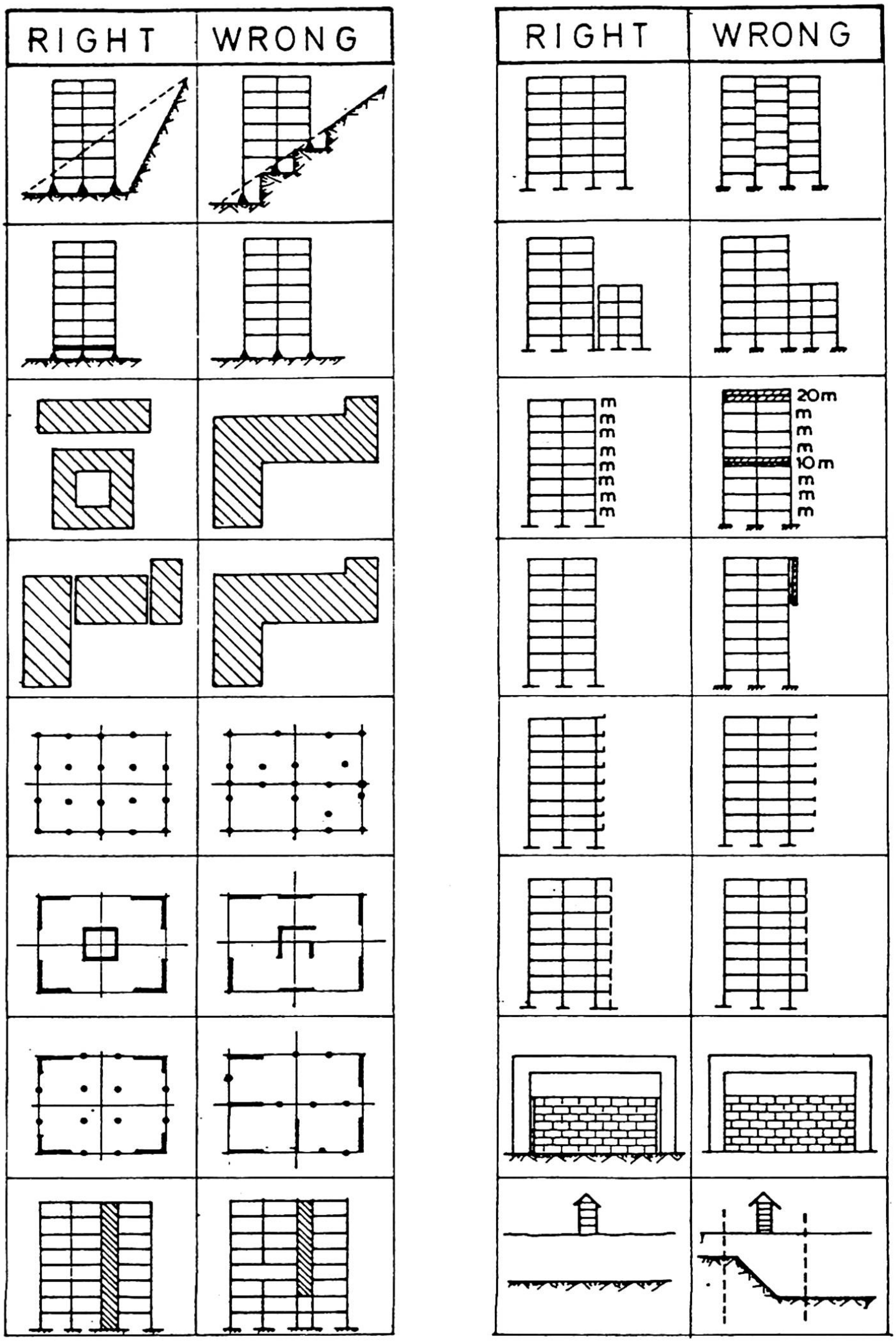


Fig. 1: Some of the rules related to the configuration of buildings in seismic zones

3.1. Structural configuration

Several Codes and textbooks provide restrictions regarding structural configuration, both in plan and in elevation; an example of those restrictions for R.C. buildings is given in Fig. 1. Some of these empirical rules are merely dictated by economical reasons against disproportionate structural costs when designing versus earthquakes. However, in their majority they are supposed to be prerequisites for the validity of the analytical models used. In fact, under the real extreme conditions of the "design-earthquake", large post-yield excursions are expected in several critical regions of the structure. Due to a large number of parameters influencing the plastic behaviour of these regions, very large **uncertainties** must be expected, which might harm (to an unknown degree) the reliability of the models used both for the analysis and for the dimensioning. Therefore, extensive asymmetries and or large non-uniformity of mass-distribution or stiffness-distribution along the structure, may render its seismic behaviour almost unpredictable. By way of example it is very doubtfull if there is any practical possibility to impart to the sections AA and BB of Fig. 2 their extremely high ductility demand.

This being said, the question arises "how much realistic are the specific q u a n t i t a t i v e restrictions set forth by the Codes" in this respect. In other words, it would be very instructive for the practical design if a rational reassessment of these rules could be carried-out, e.g. by means of several parametric studies or even on the basis of a large experience gained possibly during real earthquakes.

3.2. Stiffness of R.C. building elements, versus real action-effects under seismic loading

To the opinion of this reporter, this is in fact an everyday design problem independent of the specific structural analysis method used. Three particular cases are considered here-below, connected to this problem.

a) Justified guide-lines are needed for the selection of stiffness of the members of R.C. frame systems. Gross-section stiffness for columns is generally used, whereas beams are occasionaly considered at cracked stage.

b) A specific problem of a similar nature is raised when, applying the equivalent static planar analysis, t o r s i o n a l effects are to be introduced. If the bearing system comprises structural walls as well, the usual assumption of "column" doubly fixed at the levels of consecutive slabs is no more valid. Justified, relatively simple, artifices for hand-made calculations are welcome.

c) Finally, stiffness characteristics of coupled-walls are strongly dependent on a x i a l load (Fig. 3). Practical rules are needed both for flexural and shear stiffnesses. Systematic experimental findings in this respect will be much appreciated.

3.3. Shear strength of short columns

There is sufficient experimental and theoretical evidence (see i.a. CEB Bull. 161/1983) on the drastic reduction of both bearing capacity and ductility of relatively short columns (Fig. 4). The full M, N, V interaction proves to be very critical in case of low shear ratio values; in fact, for $\alpha_s = M : Vd$ lower than say 4, uncoupling of dimensioning for shear and dimensioning for axial actions is no more valid, even under monotonic conditions. Cyclic actions are accentuating these phenomena.

Inspite this fact, practical design rules regarding short columns are not yet included in codes. In view of the very many incidents of such building-elements (e.g. Fig. 5), further theoretical and experimental research is needed and, above all, practical guide-lines regarding the necessary modifications to be made to conventional dimensioning-methods and detailing rules of R.C. short columns under

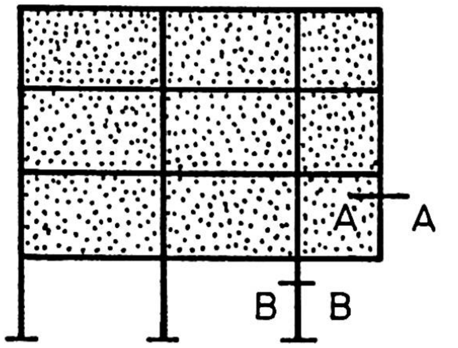


Fig. 2: This kind of structural configuration is not covered by aseismic codes; the very high strength and ductility demands of sections AA, BB reduce considerably the reliability of simple analytical models

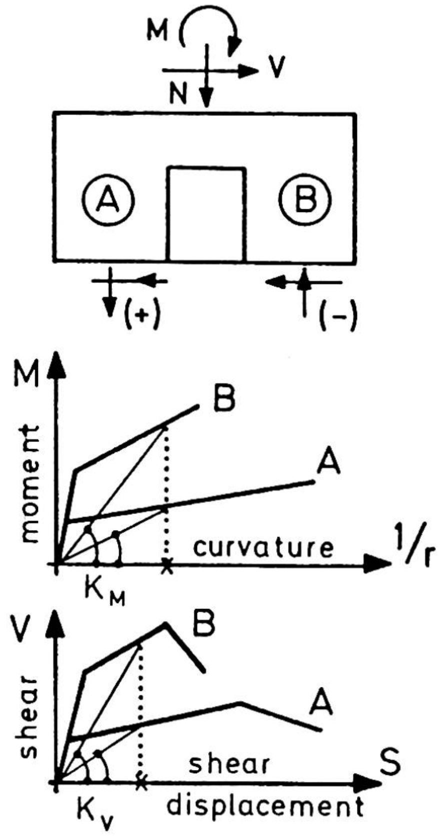
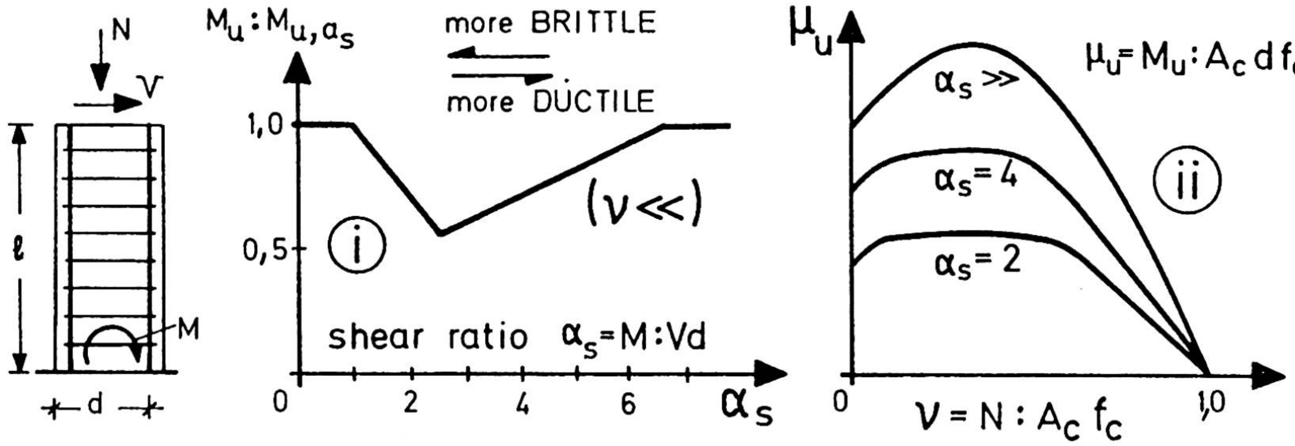


Fig. 3: Stiffness characteristics of structural walls under compression (B) and tension (A) are very much different



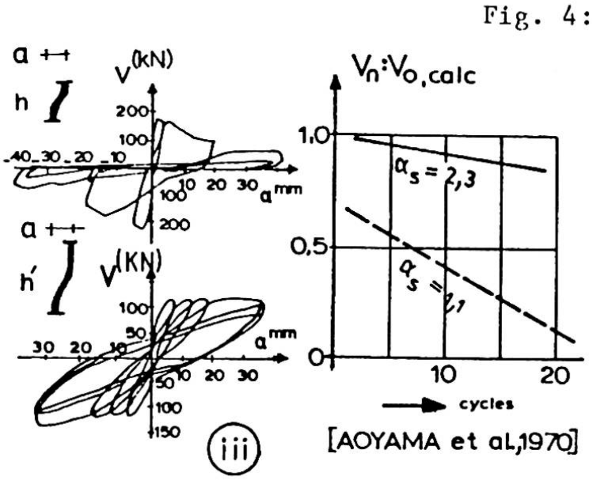


Fig. 4: Short columns (low shear ratio) exhibit:
(i) lower flexural strength
(ii) more flat M-N interaction curve
(iii) more rapid response degradation under cyclic loading
(iv) lower damping and more brittle failures

fully reversed cyclic actions. Squat-walls may be considered as presenting similar problems, and they also may be examined under this same topic.

3.4. The vulnerability of column-beam joints

One of the most vulnerable regions of R.C. frame structures in seismic situations is the column-beam joints. Their importance has been underestimated for many decades. Even now, aseismic codes provisions regarding design of joints are not internationally uniform. The complexity of the performance of these joints cannot be overemphasised.

In fact, the overall behaviour of such a joint depends on the behaviour:

a) of the anchorage of longitudinal bars of the beams, under cyclic pullouts/push-ins.

b) of the integrity of the core of the joint itself, under cyclic shear conditions.

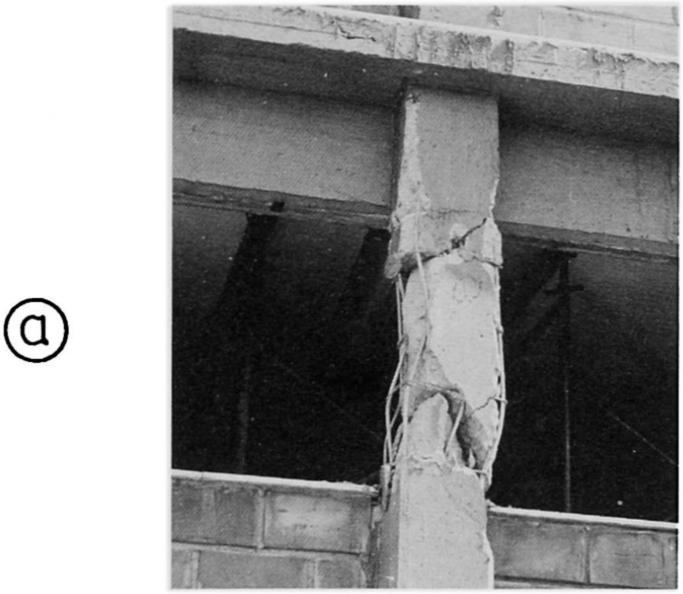
Fig. 6 is an attempt to summarise in a very schematic way the interaction between these two behaviours. Under moderate seismic conditions (or in case structural walls do not allow large displacements leading to large reversions of beam-end moments) compressive forces may be transferred through concrete after full closure of tensile cracks created by the previous cycle (Fig. 6b). In such a case, equilibrium of the longitudinal bars of the beam may be secured thanks to sufficient bond developed within the joint; transversal compression (due to the axial load of the column) is very favourable for the satisfaction of the relatively large bond demands, inspite bond degradation due to cyclic actions (compare: CEB Bull. 131, p. 74). On the other hand, shear transfer through the core of the joint is secured by the diagonal concrete **strut**, (since on each end of the strut there are available components C_b and C_c to create diagonal compression). For such a situation, two favourable consequences are derived for design: Bond may be secured without excessive additional measures, and truss mechanisms for shear transfer through the joint are not very pronounced; therefore low percentages of shear reinforcement in the joint-core are required.

However, for more severe conditions i.e. if very large displacements are imposed, without considerable redistributions of action-effects, and if a large number of full reversals is applied cyclically, tensile cracks at the beam-joint interface may not close during the next cycle; thus (Fig. 6c) compressive forces at the beam-end will be transferred to the joint only by means of reinforcement. Therefore, a cyclic pullout/push-in condition of the longitudinal bars will lead to the following doubly unfavourable result:

- Due to large reversed slips, bond degradation will rapidly take place.
- The axial force in the longitudinal bar is now almost two times higher than in the previous situations.

As a consequence, yield penetration will be rapid and the locally demanded bond (length KL in Fig. 6c) perhaps higher than available. On the other hand, since the horizontal compressive force C_b no more exists, the diagonal strut transfer of shear in the joint is alleviated and **truss** action is needed to this purpose; this action is further aggravated due to the c o n c e n t r a t e d bond forces. It becomes then apparent that if such extremely unfavourable conditions are expected, very drastic measures should be taken when designing joints: Very small diameters will be allowed for longitudinal bars, and considerable shear reinforcement will be needed in the core of the joint.

It is hoped that papers submitted in this Session of the Congress will offer criteria to assist the designer to select between the first and the second approach (which, roughly speaking, correspond to american and newzealand authors, respectively).



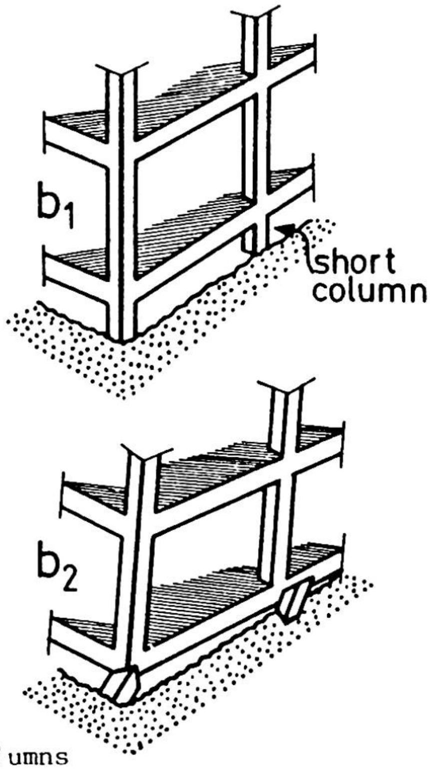
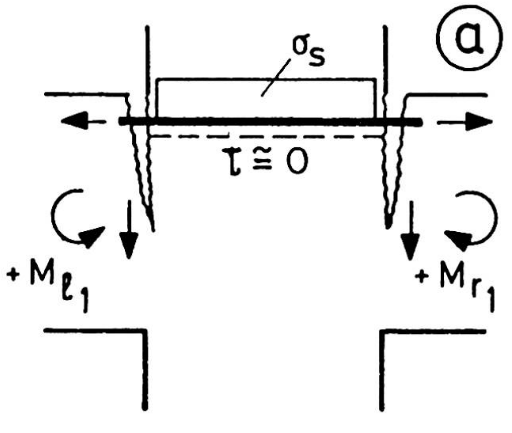
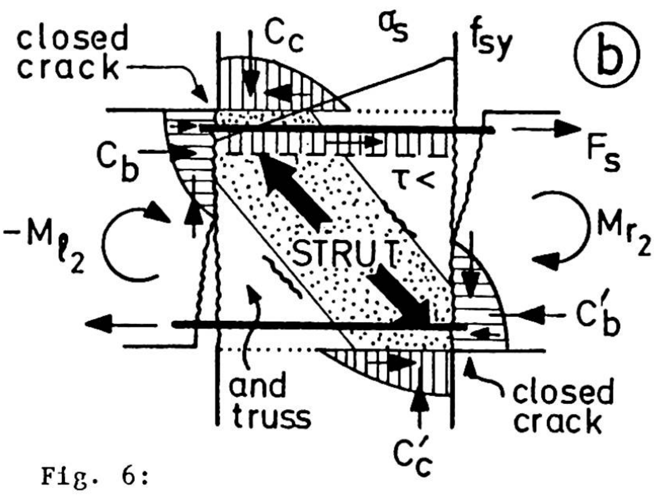


Fig. 5:

Very frequent incidents are observed on short R.C. columns subjected to earthquakes

a) The effective length of this column is drastically reduced due to the infill brick masonry
b) Typical damage in El Asnam: Failure of short columns (l = 700 mm) and almost vertical displacement of buildings





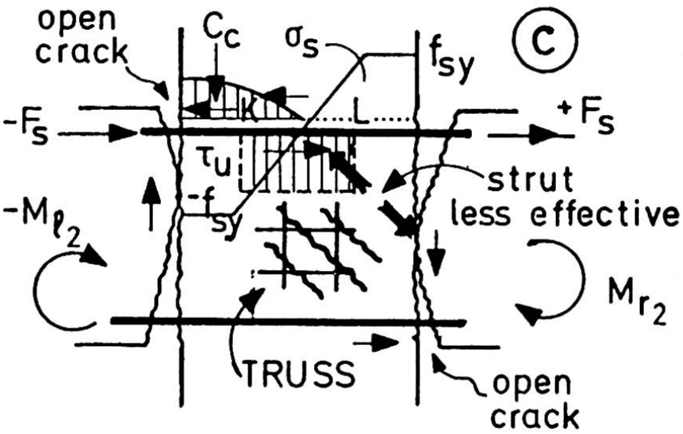


Fig. 6:

Column-beam joint (schematically)

a) Under vertical loads only
b) Under large lateral loads; previous tensile cracks at beam-ends do close during moment reversal
c) Under large lateral loads, very large displacements and large number of full reversals, previous tensile cracks at beam-ends possibly remain open during moment-cycling

3.5. Actual ductility of prestressed concrete critical regions under cyclic actions

Further evidence is needed regarding the available ductility of prestressed beam-end sections and the means of increasing this ductility. Existing code restrictions on this subject, as well as some indications that concrete confinement may not be as effective when large prestress forces are present, have produced certain difficulties in applying prestressed concrete in normal buildings situated in highly seismic zones.

3.6. Masonry

Due to the high sensitivity of beam-column joints in R.C. sway frames, reinforced masonry has recently regained its practical importance for safe low-cost housing. Full scale experiments and/or theoretical investigations on reinforced masonry walls, under fully reversed large cyclic displacements, are not as frequent as in the case of R.C. sub-assemblages.

It would be desirable to have the opportunity to read and discuss papers on this subject during this Congress. Unreinforced masonry containing appropriate strengthening-belts and ties made of R.C. are also meant to be included in this topic.

3.7. Connections of steel members

Inspite the high strength and ductility of steel per se, there is still space for additional experimental and theoretical research regarding force response and ductility characteristics of **connections** between steel elements (and specifically of beam-to-column **joints**) under large plastic reversals.

Papers on these subjects are welcome, for a better understanding of the related phenomena, in the hope to allow for less conservative design provisions.

3.8. Construction problems

Due to the additional Code requirements regarding detailing and quality of materials and workmanship used in aseismic structures, several **new** problems have to be faced during the construction in earthquake zones. Their consequences on the final performance of these structures are expected to be much more acute than in normal construction.

A couple of characteristic problems only will be mentioned here.

a) **Hoops** foreseen in the critical regions of a R.C. column (top and bottom areas) should also be provided in the core of the column-beam joint. Therefore, special techniques should be used in order to install the ready-made reinforcements of the adjacent beams. Papers dealing with solutions of this kind of problems of industrialisation of reinforcements, given in real constructions, will be useful.

b) Which specific **inspection** formats have been implemented in large and in medium size construction sites, in earthquake zones? Which are the additional organizational efforts needed and the conclusions drawn, given the sensitivity of aseismic structures versus quality drawbacks, even in the smallest detail.

Are they any field observations regarding detrimental effects of **misuse** of aseismic buildings?

Such may be the subjects of another category of papers invited in this Session.

4. CONCLUDING REMARK

It is worth to repeat here that the more or less arbitrary selection of topics proposed in the previous chapter, is by no means restrictive for the papers to be discussed during this Session of the Congress. However, if for some of these topics a concentrated effort could be given, possibly better results might be expected.

This Reporter feels already indebted to the Contributors of this Session.

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Notations

a = top displacement of R.C. column
α_s = M ; Nd =l : d shear ratio
A_c = concrete section
C_b = internal compressive force in beam
C_c = internal compressive force in column
d = height of R.C. section
f_c = concrete compression strength
F_s = steel force
l = length of cantilever column
μ, ν = normalised bending moment and axial compressive force
M = flexural moment
N = axial force
$\frac{1}{r}$ = curvature
s = shear displacement of R.C. wall
σ_s = steel stress
τ = bond stress
τ_u = ultimate bond stress
V = shear force
V_n = shear force response after "n" cycles of displacement-controlled reversals

Main Theme C2

Structural Engineering in Arctic Regions

Structures de génie civil dans les régions arctiques

Konstruktiver Ingenieurbau in arktischen Regionen

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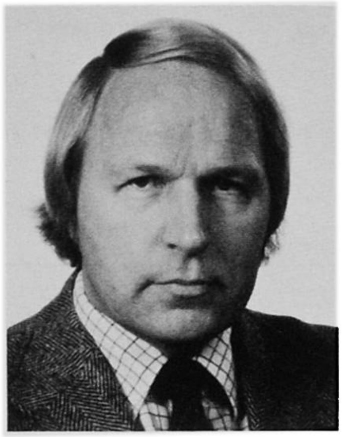

Pauli Jumppanen, born 1937, received his civil engineering degree in 1967 and his doctorate in structural mechanics in 1971 at the Helsinki University of Technology. First involved in bridge design and teaching, he came to the Technical Research Centre in 1978. He is now responsible for the research on structural engineering and design in arctic regions.

SUMMARY
The report outlines the most important features of structural engineering in arctic conditions. It discusses in general terms the influence of environmental factors on the selection of building materials and the design of load-bearing constructions. Durability problems with steel and concrete materials as well as in the corresponding structures are treated in more detail. Brief consideration is also given to structural systems and construction methods suitable for the arctic environment.

RESUME
Le rapport présente les caractéristiques essentielles des structures de génie civil dans les régions arctiques. Il passe en revue l'influence de facteurs environnants sur le choix des matériaux de construction et sur le projet des structures porteuses. Les problèmes de durabilité avec des matériaux en acier et en béton sont traités en détail de même que les structures correspondantes. Quelques considérations sont faites sur les systèmes structuraux et les méthodes de construction appropriées dans un environnement arctique.

ZUSAMMENFASSUNG
Die wichtigsten Merkmale des konstruktiven Ingenieurbaus in arktischen Regionen werden umrissen. Der Einfluss von Umweltsfaktoren auf die Wahl der Baustoffe und den Entwurf von Tragwerken wird generell erörtert. Anschliessend werden eingehender Probleme der Dauerhaftigkeit der Baustoffe Stahl und Beton sowie damit erstellter Bauwerke behandelt. Zum Schluss wird noch kurz auf die für arktische Regionen geeignetesten Tragwerkssysteme und Baumethoden eingegangen.

1. INTRODUCTION

Several definitions related to the scope of interest have been given to arctic regions around the North Pole. Definitions based on average temperature or the warmest month of the year as well as on the boundaries of the forest zone or the northern lights region have been used in natural sciences. Very convenient limits from the technological point of view are the boundary of the permafrost area on the land and the maximal extension of the ice cover on the sea. According to this definition, more than 60 % of Alaska, about the half of Canada and the Soviet Union, and some parts of Scandinavia and China are included in the arctic region (Fig. 1).

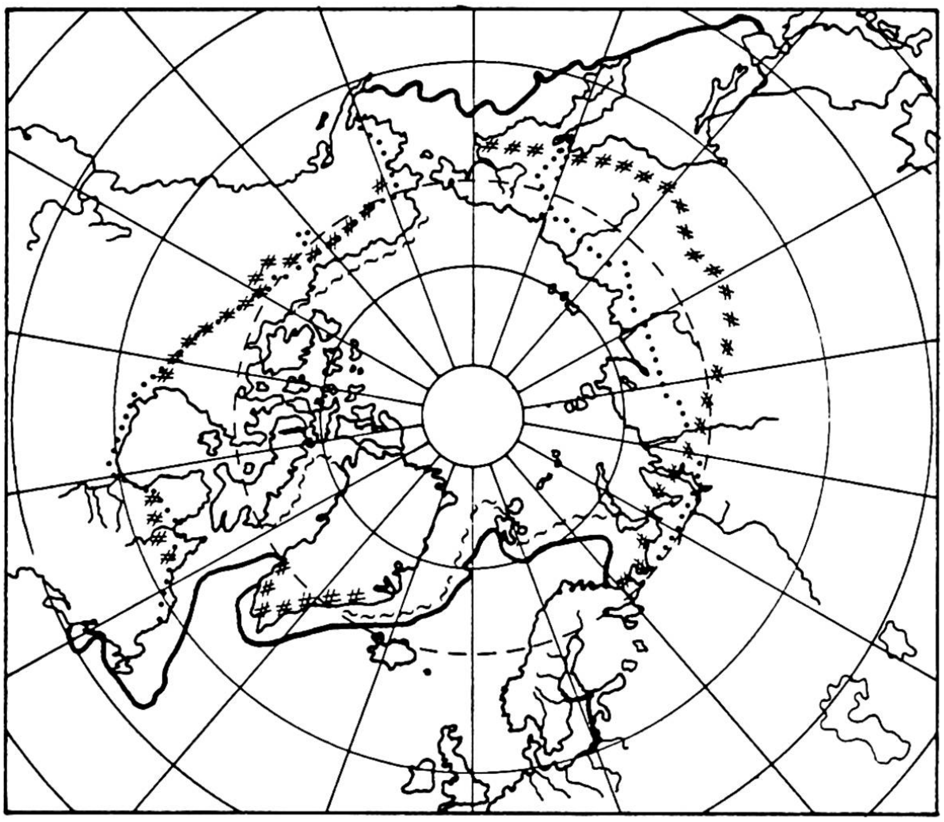

Fig. 1. Definitions of the arctic region from the technical viewpoint:
······· boundary of forest zone
boundary of permafrost area
~~~~~ boundary of arctic water region
—— average maximal extension of sea ice.
~~~~~

In the so called subarctic regions, where the cold climate, frozen ground and the appearance of heavy snow cover cause difficulties for building and transportation activities, the problems and also the technical solutions are to a large extent similar to those in arctic conditions. As far as the antarctic region is concerned, the environmental factors are still more extreme than in the Arctic. The technology developed for arctic applications can form a starting point for the future utilization of natural resources in the Antarctic.

Traditionally, fishing and whaling have been major activities in arctic waters. In the nineteen fifties and sixties, the development of the forest products industry and the building of hydroelectric power plants gave an economic stimulus in arctic regions. In the mid-seventies, the dramatic increase in the price of oil changed the situation totally. The exploration for arctic oil and natural gas began on a large scale. More extensive mining of coal in northern Canada and the Soviet Union is also envisaged. Mining metals such as copper, gold, nickel, lead, silver, zinc, cadmium, titanium, etc. and other minerals and building materials such as asbestos, limestone, pyrophyllite, shale, gypsum etc. has been a traditional activity in certain parts of the Arctic. The importance of mining is increasing rapidly because of the industrial need for some important raw materials and a special need for certain strategic metals. Mineral resources both in the permafrost areas and on the continental shelf will be exploited on a large scale in future decades.

The exploration and the exploitation of arctic energy, mineral and material resources require many kinds of building operations on the land and in the sea, e.g. building of roads, bridges, factories, power stations and harbours as well as facilities for the exploration, production and transportation of oil, natural gas, coal and minerals. It is a challenging task to carry out building projects in the Arctic from the viewpoint of both technology and management. The problems orginate from the cold climate, the presence of ice, snow and permafrost, long distances and inadequate transportation network, lack of local labour and building materials, the fragility of the nature, and from many other of factors varying importance. Technological difficulties and strict safety requirements demand special care in the planning of building operations and in the design of structures.

The purpose of this paper is to give a short introduction to basic problems in the design of structures and the selection of materials to be used in an arctic environment. A brief mention is also given to environmental factors and their influence on construction methods in the Arctic.

2. ENVIRONMENTAL CONDITIONS

2.1 Cold climate

Cold climate influences materials and structures as well as practical construction work. In winter, the lowest temperatures in the continental regions and on the arctic islands can be below -60^{o}C. The temperature can stay below -50^{o}C for several weeks at a time. Since the summer temperature can reach values between +30^{o}and +40^{o}C, maximal temperature variation in some areas can be close to 100^{o}C.

On the coast and in sea areas, the climate is milder and the lowest temperatures in many places are between -30^{o}C and -40^{o}C. The working conditions are made difficult by strong winds and the combined effect of wind and low temperature. This effect is measured by the so-called windchill-factor. Values of the factor are given in several handbooks and publications. For instance, the temperature -40^{o}C and wind speed 0,5 m/s has the same effect as the temperature -7^{o}C with wind speed 20 m/s.

2.2 Snow and ice

Amounts of rain in the Arctic, especially in the tundra region, are quite small. Similarly, the snow cover is in general thin and the snow loads on roofs and horizontal structures somewhat less than in the subarctic region, in most areas 70 kg/m^2 or less.

The main problems are caused by moving snow and the accumulation and packing of snow against buildings or in other special places. The transportation of snow starts when the wind speed is more than 10 m/s and increases rapidly after the speed 15 m/s. The density of pack ice can reach the value 300...500 kg/m^3, and in multiyear snow fields the value 700...900 kg/m^3 which is close to the density of ice.

On the sea, ice loads and their effects raise the most serious problems for structures and building operations. Ice loads are caused by the movement of ice cover (thermal, wind, currents), first-year or multiyear pack ice layers and ice ridges, and floating icebergs. The maximal thickness of ice cover on arctic seas varies from one to three metres, the thickness of pack ice from 3 to 5 or more metres, and the thickness of ridges up to 50...70 metres.

Although ice conditions and ice loads on structures have been studied by numerous researchers and institutes in recent years, many unsolved problems remain, such as loads caused by multiyear ice, ice loads on large scale structures (harbours, artificial islands), and problems resulting from ice jams and break-up of ice in arctic rivers.

The development of ice and snow control, management systems for building sites, and large field operations are also needed. This is necessitated by differences in local conditions and possible rapid changes in weather. The systems can be based on weather forecasts or on special satellite, flight and field observations. The local control can be based on field measurements, special geometry of buildings, protective structures etc.

2.3 Permafrost

The most dominating phenomenon when building on land in the Arctic is the permafrost. The thickness of the frozen layer varies from a few metres to as much as 1500 metres. The permafrost can be continuous, discontinuous or composed of several types of local formations, such as of pingos and palsas.

The soils on the surface melt annually to a depth which can be evaluated from the formula (z in cm)

$$z = k \cdot \sqrt{\sum_i h_i T_i}$$

where h is the time (in hours) and T the temperature (in ^{o}C). $\sum h \cdot T$ is then the time-temperature amount of the warm period of the year and k the coefficient depending on the soil type and surface vegetation. k varies from 0.5 to 1.5. The depth of the melted layer, the so-called active layer, is between 0.5 and 2.0 m in a continuous permafrost and between 0.7 and 5.0 m in a discontinuous permafrost.

The appreciable strength of frozen soils can be utilized in many ways in building operations in the permafrost area. The problems are basically caused by the melting of frozen soils. The active layer is typically very moist and has a very low load-bearing capacity. Also a more extensive melting of permafrost can take place during a warm period e.g. in the case of flooding. The harmful effects of soils produced by melting are the thermokarst process, which maintains a continuous melting process in the permafrost, thermoerosion and material transportation with running water, and local or even larger earth-slips in the ground.

2.4 Other environmental factors

In addition to the factors mentioned above, arctic regions have many other special features which need to be taken into account in the design of structures and the planning of construction works. Such features include:

- Low visibility and darkness caused by the long winter period, fog, seedust, and moving snow.

- Restricted variety of local building materials available.

- Erosion and transportation of soil materials due to rivers, floods and sea currents.

3. BUILDING MATERIALS

3.1 Local materials

In arctic construction building materials are often used on a large scale. Examples of very large constructions are platforms and artificial islands used in the exploration and the production of oil and natural gas, pipelines, industrial plants, roads and bridges for large loads, power plants etc. The need to utilize local materials wherever possible is therefore obvious.

Ice cover in seas and rivers can be used for transportation, and also some building works can be carried out using the bearing capacity of ice. The thickness of the ice cover can be increased e.g. by pumping water onto the ice or by using cooling machines. Ice can also be reinforced by timber products, steel bars or cables, plastic fibres etc. Also for road and foundation construction the use of ice as the natural or reinforced material appears to have some potential.

Sand and gravel are generally obtainable from river banks and the sea bed. These materials are needed especially for roads, foundations, harbour structures and artificial islands. Their use both in a non-frozen and a frozen condition is possible. Sand and gravel can also be used for making concrete in the Arctic, and local aggregate materials often give very good durability in concrete structures.

The availability of mining slacks as well as crushed stones and rocks will increase as mining activities increase. Materials of this kind can also be used for concrete aggregates and road construction.

There are extensive timber resources at low latitudes of the permafrost areas. Several kinds of raw materials for the building industry can be obtained from the arctic ground. However, the manufacture of building products and components usually takes place in industrial plants outside the arctic region.

3.2 Materials of load-bearing constructions

Basic materials in arctic construction are the usual steel and concrete. Advantages of steel are good availability, adequate range of special steel qualities, possibilities of using both welded and bolted connections, and finally low price. The main problem in steel construction arises from the brittle fracture behaviour of steel at low temperatures. Another serious problem is the corrosion which takes place, especially in the aggressive sea climate.

The toughness of steel is generally measured by the well-known Charpy V-test. By means of measured energy absorption values, a fracture toughness curve is drawn and the so-called transition temperature between brittle and plastic fracture behaviours is determined. For structures in an arctic environment subjected to dynamic loading (e.g. bridges, vessels, platforms), the steel material must satisfy the fracture toughness requirements at temperatures between -40°C and -60°C.

The toughness of a steel material can be increased

- by decreasing the strength and the grain size of the steel and

- by using magnesium, nickel, copper, chrome and possibly some other ingredients in the steel.

The best structural steel in welded constructions for an arctic environment is often fine-grained medium-tensile steel with high fracture toughness values (Fig. 2).

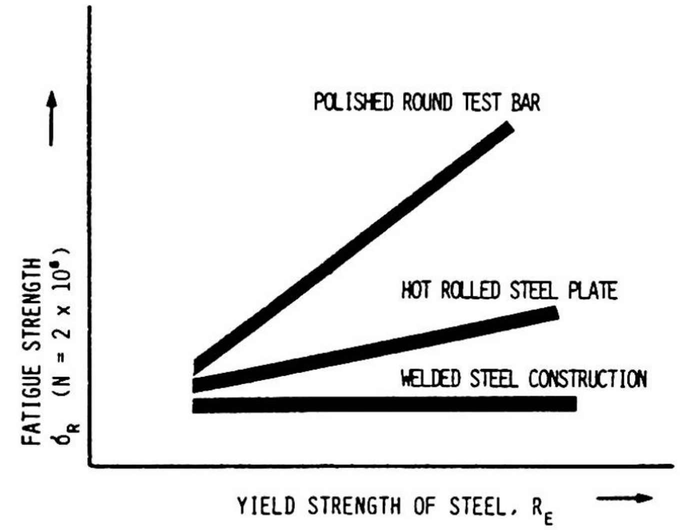

Fig. 2. Interaction between fatigue strength and yield strength of steel. The picture shows that high yield strength values cannot be utilized in welded constructions.

The advantages of concrete in arctic construction include the possibility of using local raw materials, sturdiness and suitability for gravity structures especially in the case of ice loads, good protection against wearing and corrosion, and high fire resistance qualities. In construction projects, the use of either prefabricated structural members or in situ concrete is possible.

The strength of concrete generally increases with decreasing temperature (Fig. 3). The basic problems with concrete construction are concreting works in the cold climate and the durability of concrete in the arctic environment. Special methods for making concrete at low temperatures are often based on heating and protection and on the use of some admixtures. The aggregate and the water can be preheated up to +70^{o}C before the concreting. The temperature of fresh concrete during casting must be between +5 and +40^{o}C. The cast concrete members must be heated and protected until a required strength against the freezing is reached. The admixtures most used for better handling of fresh concrete are freezing point reducing agents and plasticizers. Accelerators can be used for reducing the need for protection.

The main reasons for durability problems with concrete are the freezing of fresh concrete before hardening, the effects of the freezing-melting process in the long term, and the corrosion of concrete especially in a sea environment.

The freezing of fresh concrete causes some expansion of 2 %. The accumulation of ice inside the structure and a considerable loss of strength are possible. Large variations in temperature and the humidity cause a repeated freezing and melting process in concrete. Several freeze-thaw cycles can easily damage the surface parts of a concrete structure. Resistance to freeze-thaw behaviour is based on the optimal porosity of concrete which allows the movement of moisture and the expansion of freezing water in pores. The porosity can be increased to a required level by using air-entraining agents in the manufacture.

Corrosion risks of concrete appear mostly in the sea environment and are caused by clorides, sulphides and carbon dioxide in sea water. Also bridges and other structures in rivers are subjected to corrosion to some extent. The corrosion usually starts from small cracks and leads in the first stage to some deterioration of the concrete surfaces. The corrosion of reinforcing steel takes place in the second stage and results in some loss of safety in the long term.

Resistance to corrosion can be increased in several ways. Some possible methods are increasing the tightness of concrete by prestressing, the use of special materials and ingredients in the concrete mix (e.g. special hydraulic cement) and the use of non-corrosive or specially protected reinforcing steels. The use of copper or zinc has been considered in this connection.

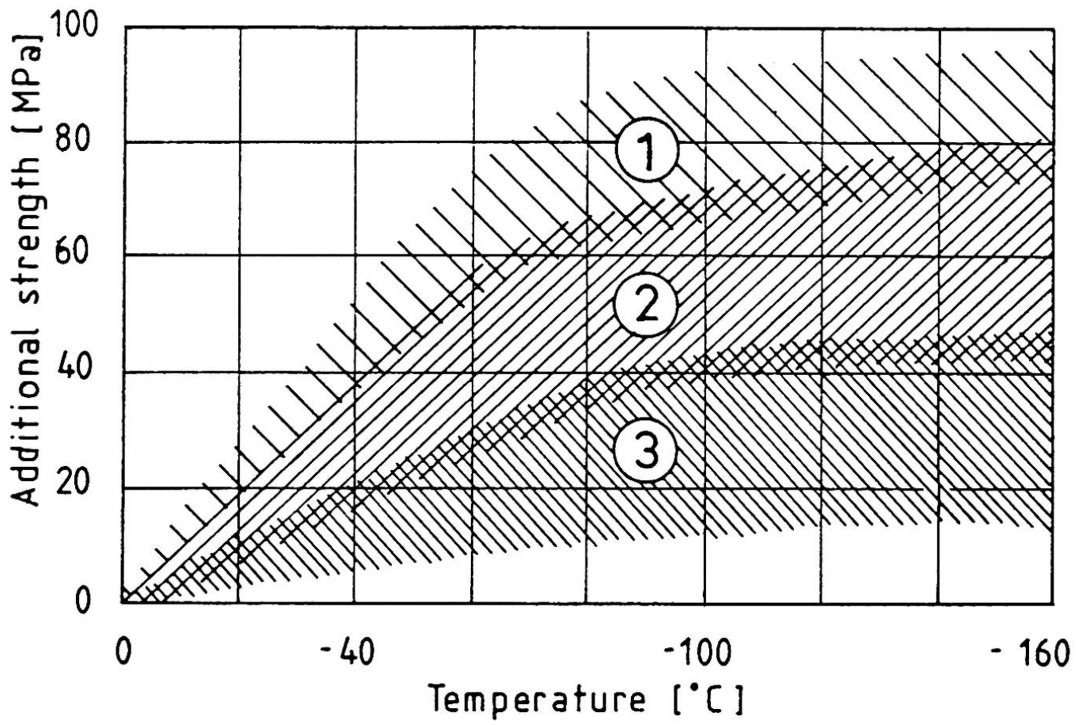

Fig. 3. The effect of temperature decrease on concrete strength in the case of 1) wet concrete 2) moist concrete and 3) dehydrated concrete (+105°C).

3.3 Other building materials

There are several other building materials in addition to steel and concrete which can be successfully used in the arctic climate. These include wood, aluminium, brick and block products and some plastics. Insulation materials for thermal, frost and acoustic insulation as well as coatings and protection materials are also needed on a large scale. These are not considered in this paper.

Wood retains its mechanical properties very well in the frozen condition. Also glued wood products, such as plywood and glued beams and arches, can be used if the glues used in their manufacture are suitable for the required temperature and moisture states. Timber constructions are mostly used for housing. Application in small bridge construction could also be possible.

Aluminium would be an ideal material because of its good processability, high strength values, and high corrosion resistance. The large-scale use of aluminium is prohibited by its high price. Possible objects of use are light gauge plates and profiles for buildings, light offshore modules, mast structures, storage tanks etc.

The applicability of bricks, blocks and other light-weight material products is quite limited, there being problems connected with possible moisture and frost damage. Therefore the strength values as well as porosity and moisture properties should be checked before use. On the other hand, the use of light-weight concrete has aroused quite a lot of interest because of the advantages in transportation both on land and sea.

4. DESIGN OF STRUCTURES

4.1 Loads and actions on structures

The load-bearing structures are normally dimensioned for their own weight, service loads, and wind and snow loads. In the case of harbour and marine structures, also the effects of waves and currents are to be considered. In arctic regions several other loads and effects must be taken into account. The following list contains the most important factors for the design of structures in the Arctic.

Ice loads are of primary importance in the design and dimensioning of harbour and off-shore structures as well as of bridges. The basic load types are:

- static loads from the pressure of ice cover and ridges

- dynamic loads caused by moving ice and the combined effects of ice and waves

- impact loads from floating ice and icebergs.

Other effects caused by moving ice are wearing and scouring phenomena. Scouring of the sea bottom and marine foundations is often caused by movements of icebergs and large ridges. In the most serious cases the scores can have a depth up to 20 m. Worthy of mention is still flooding of rivers raised by the break-up of ice or ice jams. The combined effects of running water and ice often cause very high loads on bridge structures.

Special influences on arctic design also result from the low temperature values and the large temperature variations as well as large differences in the summer and winter temperatures. Material effects which have to be considered are the brittleness of metals and the freezing as well as the freezing-melting process in porous materials. In strength calculations, large thermal stresses especially in massive structures must be taken into account.

4.2 Foundations

In the building of foundations on land in the Arctic, the main problems arise from the yearly melting of permafrost, risk of additional melting caused by the heating of structures, movements of soils, and extensive floods.

The design of foundations can be based on the following three principles.

- Utilization of zones free of permafrost where the stability and the load-bearing capacity of soils are adequate.

- Maintaining the soils in a frozen condition by the use of special constructions.

- Allowing the melting of frozen soils up to a certain dimensioning depth.

The second alternative is maybe the most common in arctic construction and is generally based on the use of pile foundations. The piles are driven in to the required depth (3...5 m) inside the permafrost and a free air space between 0.5 and 1.5 m is left between the structure and the earth's surface. Special pile types, where the heat flow along the pile is prevented, have also been developed. Another method, specially applied to light-weight structures, uses a sufficient layer of some insulation material under the structure.

The third alternative method mentioned above can be used only in special cases. If a soil has a very good load-bearing capacity and small deformations in the melting and freezing process, or if there are rocks or other stable solid layers close to the earth's surface, the melting up to some calculated depth can be allowed.

4.3 Structural systems

Ice conditions and ice loads play a very dominant role in the planning and design of marine structures. Ice phenomena in rivers, such as effects of ice jams and break-up of ice, also have appreciable influences on bridge design. When building on land, the main problems are not associated with structural design. Basic problems are generally involved with the material selection, the transportation, and the construction methods on the building site. This fact has had a significant influence on structural systems used in the Arctic. Instead of building on site methods more advanced systems consisting of prefabricated elements, space units, small- or large-scale modules have been developed in great numbers.

Steel structures have very good prospects in arctic construction. Typical applications are caisson-type structures for harbours, terminal structures, oil and gas pipelines, factory buildings, warehouses, towers, buoys, masts, bridges etc. Construction units of steel can be manufactured in a factory, transported to the building site, and assembled by welding or bolting. A trend towards using very large modules with a high stage of prefabrication is obvious e.g. in the building production plants, factories and office buildings.

The design of arctic steel structures is quite similar to that in the sub-arctic climate. In the selection of steel material for welded and dynamically loaded constructions, the toughness criteria must be taken into account. Careful dimensioning should also be made for the fatigue of the total construction and for the stresses caused by the combined effects of snow and wind as well as by large temperature variation. High-quality corrosion protection is also required especially in the aggressive sea climate, because the preparing of the protection in an environment like this is sometimes quite impossible.

Concrete structures have certain special advantages in arctic construction as mentioned earlier. Typical objectives of use are caisons and other harbour structures, foundations in general, industrial plants and power plants, storage buildings, bridges etc. Both construction with on site methods and prefabricated construction units can be used in building with concrete. In the former case, the compatibility of water and local aggregates should be tested beforehand.

In the design of concrete structures, special attention should be paid to the corrosion resistance, the fatigue, and the thermal stresses in solid structures. Corrosion protection can be based on the high quality and the successful selection of materials of concrete mix, optimal tightness of protection layers, prestressing, and the protection of reinforcing steels. Although the fatigue properties of concrete structures are in general good, extensive diagonal cracking and low-cycle fatigue failures can sometimes take place especially in the presence of cyclic shear and torsion. Brittleness of reinforcing steel can in such a case cause damage to the total structure.

Thermal stresses in massive structures are usually larger than e.g. in steel structures. In concrete structures, the highest stresses are appreciably reduced by the creep of material. On the other hand, continuously repeated creep effects cause forced stress in the structure. Very careful study of the effects arising from low temperatures and large temperature variations is therefore to be recommended.

Main Theme D

New Frontiers in Structural Engineering

Les nouvelles frontières du génie des structures

Aufbruch zu neuen Grenzen im konstruktiven Ingenieurbau

Leo FINZI
Professor
Technical University
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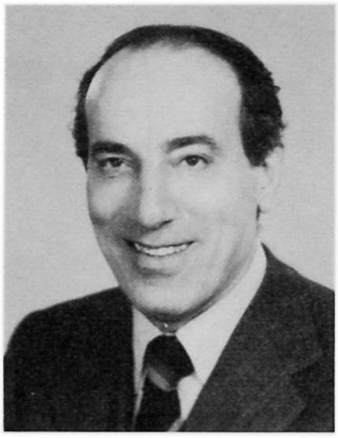


Leo Finzi, born 1924, has been a full professor of Theory of Structures since 1958. Author of papers on elastoplasticity, stability and dynamic analysis of structures, he has been and is the designer of outstanding steel and concrete structures.

SUMMARY
This general report is intended to make engineers aware of what is really inventive today in structural engineering (new types of structures, new ideas and methods in design, fabrication and erection) and what could be expected in the „nineties" i.e. in the near future.

RESUME
Ce rapport devrait permettre aux ingénieurs de prendre conscience de ce qui est vraiment inventif dans le génie des structures: nouveaux types de structures, nouvelles idées et nouvelles méthodes de projet, de fabrication et de construction. Il présente les tendances possible pour les années „quatre vingt-dix".

ZUSAMMENFASSUNG
Der Bericht hat zum Ziel, den Ingenieuren bewusst zu machen, was im Bereich des konstruktiven Ingenieurbaus heute wirklich neuartig ist (neue Arten von Tragwerken, neue Ideen und neue Methoden des Entwurfs, der Fabrikation und der Konstruktion) und was für die 90er Jahre — für nächste Zukunft also — erwartet werden könnte.

1. INTRODUCTION

In order to try to foresee what tomorrow's structural engineering may be like, it would probably be best to analyse the new ideas that already exist today, the new technologies that are about to emerge from the experimental stage. It would also be necessary to consider what might be the requirements of our society in the near future, and so what might be the demands made on tomorrow's engineers. To do this does not imply that we have to become visionaries gazing into the next century, but we can discuss what may lie in store for structural engineering in the next decade.

Structural engineering evolves and progresses together with the evolution, progress and refinement of:

- materials
- structural components and fasteners
- structural shapes
- physical, mathematical and numerical models
- fabrication techniques
- erection techniques

One of the purposes of this introductory report is to draw the attention of possible contributors to this session of the 12th IABSE Congress to possible or desirable innovations in the fields just mentioned, in the hope that they will then give us the benefit of their ideas and proposals.

In other words, our intention here is to encourage architects, engineers and, in general, those involved in this field, to present papers on whatever there is that is really new and inventive (new types of structures, new structural concepts, new fabrication and erection techniques), and on whatever might reasonably be expected in the near future. Even ideas which today seem utopian may in fact soon become a matter of common practice.

2. MATERIALS

Materials now exist which have already made great technological advances, such as structural steel and concrete. Others, such as plastics and aluminium alloys, still have not overcome all the obstacles facing them in terms of their structural application - obstacles of some importance from the point of view both of their reactions to the environment (temperature, weather conditions etc.) and of their production costs. Even for the well-tried traditional materials there are still good hopes of progress.

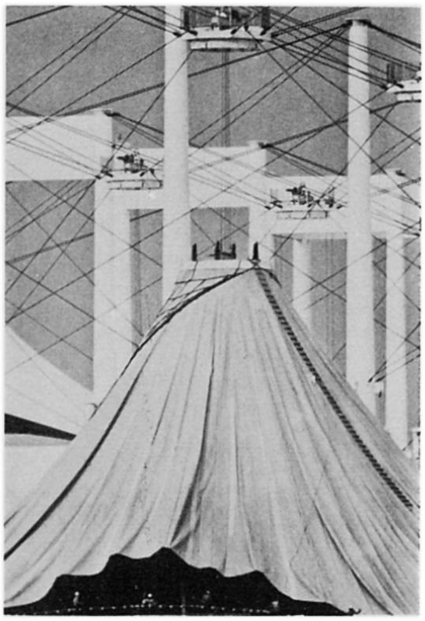


Fig. 1 Jiddah air terminal under erection

Structural engineers, for example, still reasonably hope for improvements in steel to deal with problems of elastoplastic instability. Of course, it would be best if this could be done by increasing the values of Young's modulus through metallurgic operations on the orientation of the crystal lattice, but if not, at least by developing manufacturing processes to reduce to a minimum the residual stresses, which panalise metal components in general and columns in particular |1|.

Would it not also be reasonable to hope that present studies on various types of fibre reinforced concrete may eventually give us a material able to cope not only with compression, but also with tension and shear?

Increasing energy costs do not leave much room for hope in the increased use of aluminium alloys for structural engineering, at least in the near future. The situation may be more hopeful in the field of plastic materials, whether used for wires or cables, profiles or even for two-dimensional sheets (Fig. 1).

It is well known that for large span structures a decisive parameter is the critical length L_{cr} of the material involved which is the ratio between the yield strength F_y and the specific weight γ (i.e. $L_{cr} = F_y/\gamma$).

A characteristic of plastic materials is that their specific weight is of the order of one. Since the yield strength is not much less than for standard steel, these materials seem to have advantages where the dead weight of structures plays a dominant part.

But the dead weight may also be decisive for large span reinforced concrete structures. Here, great progress may be achieved if really lightweight concretes can be obtained (i.e. with less than half the specific weight of normal concrete) without too much reduction in the compressive and tensile strength and the elastic modulus.

There may be many ways to obtain this desirable result: the use of light aggregates with expanded clay, the use of concretes incorporating expanded polystyren balls, the air-entrainment of the concrete. Each of these different possibilities involves differences in mechanical behaviour, which should receive very careful attention, since the well known characteristics of ordinary concrete can certainly not be extrapolated to these newer types.

Masonry of bricks and mortar was all very well when man's principal tools were his hands, but now it must give way to new mixtures which offer the same possibilities for insulation, but which can be dealt with by machines.

One result is much greater freedom of choice for size and shape: panels, folded beams, entire box units can be cast to obtain monolithic elements that may be even very large and have the desired characteristics not only of strength but also of insulation.

When discussing construction materials, the foundation soil must not be forgotten, since it is one of the essential components. The type of soil is an essential factor, decisive for the choice of foundation systems in general, above all, of course, for underground work such as tunnels or wells. Here, the way that the soil is treated to render it as suitable as possible for particular requirements is really fundamental.

Foamed cements, acrylic fibers, epoxy resins already make it possible to transform otherwise totally unsuitable silts into something even better than sand or gravel.

3. STRUCTURAL COMPONENTS AND FASTENERS

The sinking of suitably shaped reinforcing bars into concrete castings and the use of tendons for prestressing led to completely new structural components.

Another breakthrough was the design of large steel beams made up of plates suitably strengthened by longitudinal and transversal stiffeners. Even wood, perhaps the oldest of all structural materials, has been improved by lamination to remove its anisotropic defects.

What may be the next developments for the immediate future?

The growing use of meshes and textile materials perhaps even together, seems very promising, particularly in terms of high performance versus low dead weight (Fig. 2) |2|. Glassfibre - reinforced polyester resins have also by now established a place for themselves in the construction of light and efficient

Fig. 2 U.S.A. Pavillion air supported roof at Osaka Expo 70

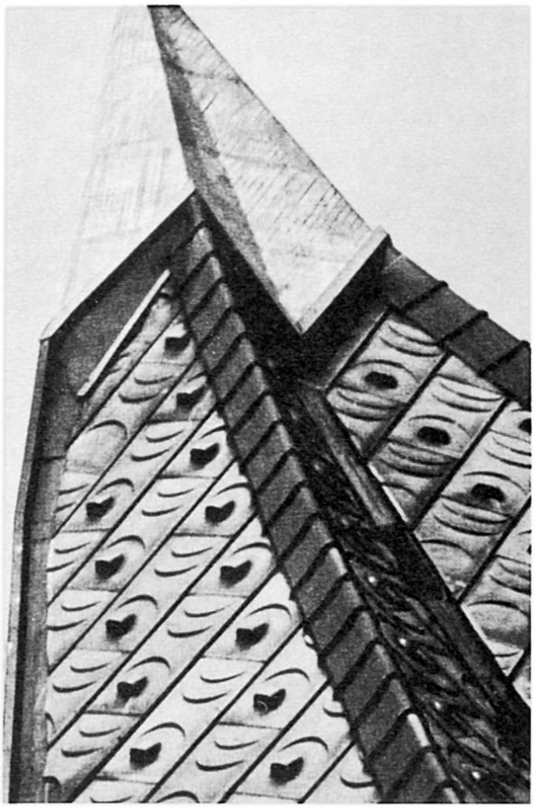


Fig. 3 Fiberglass reinforced polyester panels

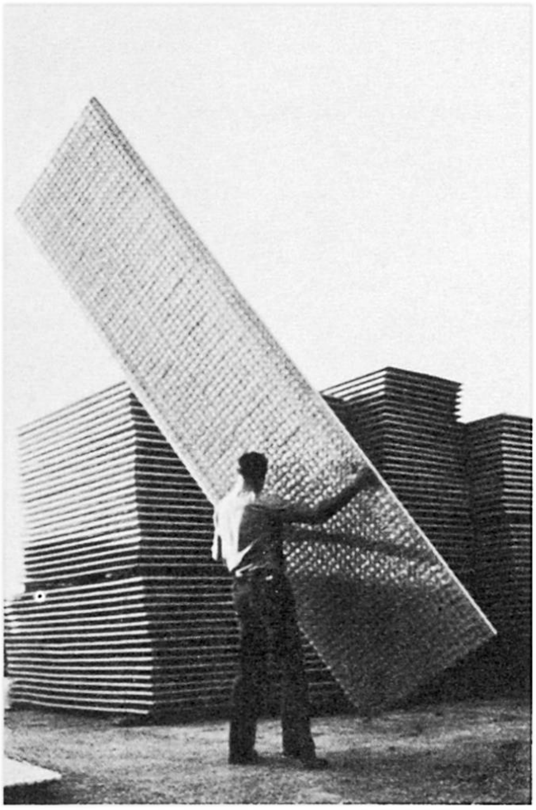


Fig. 4 Extralight steel reinforced panels before cement gun spray

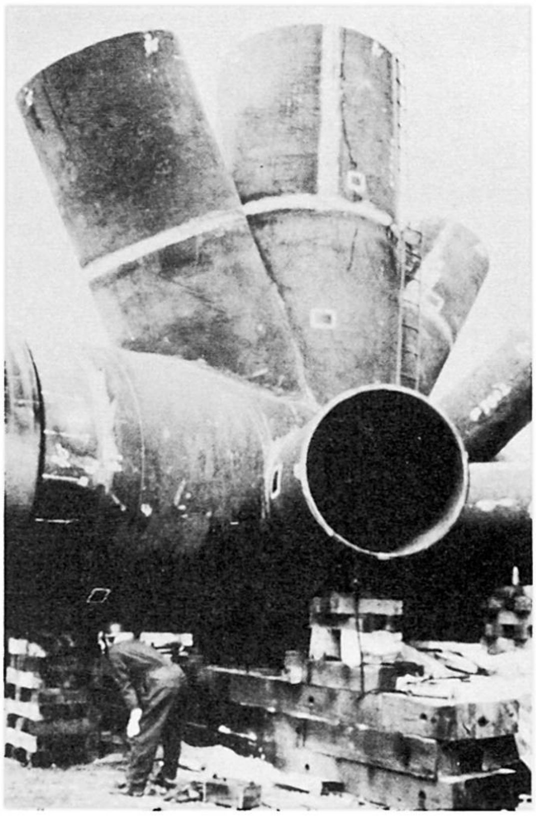


Fig. 5 Jumbo offshore truss welded connection

structural components (Fig. 3) |3|.

For smaller spans there are now sandwich constructions which incorporate within the insulating core a light steel space frame to connect the external concrete layers. They have proved to be particularly suitable for low cost housing (Fig. 4).

The rapid development in the use of macrostructures must also be considered, not only for large offshore constructions but even for residential purposes. This has led to notable progress in the construction of components of exceptional length and width. Tubular constructions seem to offer the greatest advantages here, to deal with problems of statics and the technological difficulties involved. However, the dimensions of these sections are so great that it seems better to think of them in two-dimensional terms, rather than as traditional one-dimensional elements (Fig. 5).

The combination of steel with concrete has had great success in structural engineering in various forms, from the traditional reinforced and prestressed concretes to composite steel-concrete structures (steel beams with reinforced concrete slabs, corrugated sheeting combined with concrete filling). Might it not be possible to combine other materials equally successfully, such as alluminium alloys, wood, nylon and so on?

Perhaps even more impelling is the need for innovation in the field of connections.

Steel member connections and the joints of corrugated rebars in reinforced concrete must be capable of simplification. I am, of course, referring to connections made on site, since fasteners manufactured in shop or prefabrication centres can already profit from automatic welding processes and rebar connections of the Bar Grip, Lenton or Cadweld type that have already been highly simplified and rationalised.

Will glued connections have a prosperous future?

The example of the aeronautics industry might lead to an easy optimism, but it would be well to remember that an open building site is not quite the same thing as a Boeing aircraft plant.

Nowadays, the impulse towards the highest possible degree of simplification for in-situ joints has led to favouring hinged type buildings almost everywhere, whether in steel or reinforced concrete. However, it may not always be the right answer, because not only does it imply giving up the economic advantages deriving from continuity between structural members, but it also leads to structures that are not ductile enough to resist earthquakes, and that may be subject to progressive collapse as a chain reaction to local damage.

Will it be possible to make subsequent modifications to a joint, when the structure has already been erected, to ensure full continuity between the members meeting there? Work of this kind has already been done on some offshore constructions |4|.

4. STRUCTURAL SHAPES

The last decades have been so richly stocked with innovations in the realm of structural shapes that it might seem that there is little more to be said. Shells, folded plates, hanging roofs, space trusses, box structures have all been widely used for constructions of considerable importance. The designs of high-rise buildings have also changed, particularly in the bracing systems, and can now reach heights of over 400 m and eliminate the intermediate columns at each floor |5|. As to bridges, the introduction of box girders with orthotropic plates, the use of straight stays for spans of hundreds of metres, and the capacity to control aeroelastic instability over exceptionally long spans through suitable expedients have permitted the design of some constructions of great beauty and originality.

It seems to me, however, that stimulating new problems are about to be posed. Look at a nuclear power plant, normally close to the sea or a large river, rising on ground that may be none too reliable, enclosed within massive walls rather like some medieval castle. Then think how much more interesting it might be to construct floating power plants anchored offshore or even under the sea. The offshore drilling platforms (Fig. 6) already allow for immerse containers located at as much as 500 m in depth. As in the case of Mahomet and the mountain, if the water will not come to the power station, we can take the station to the water.

As to more normal buildings, it is true that we have learnt to go up to over 100 floors, but why not abandon the traditional idea of isolated blocks and connect them together in macrostructures that will join up working and residential requirements with the road system of the city.

Cable and inflatable structures, with many excellent examples already in existence, lead one to consider coverings for large areas. It is thought, for example, that a translucent plastic dome 2 cm thick set up over a diameter of 2 km (Fig. 7) and with a maximum height of 400 m (i.e. covering an entire city center) could tollerate an internal overpressure of 1500 Nm^{-2}, quite enough to compensate for any normal actions (such as snow and wind) due to weather conditions |6|.

The surface of the earth has large deserts burnt by the sun all through the year, where regulating the climate in inhabited areas is of fundamental importance. The area available is vast, with inexhaustible solar energy on the spot. Structural engineering can help to solve the problem, and Schleich's ideas (Figs. 8 and 9) |7| on the subject are the result of his brilliant work on cable-net cooling towers |8|.

Man's needs are changing more and more rapidly, and to deal with this situation of great mutability, structures are required with the smallest number of constraints to spatial distribution, both for industrial and residential buildings. All of this tends to lead towards long spans. Large public works, too, show the same trend, road systems with the relative bridges, tunnels etc.,

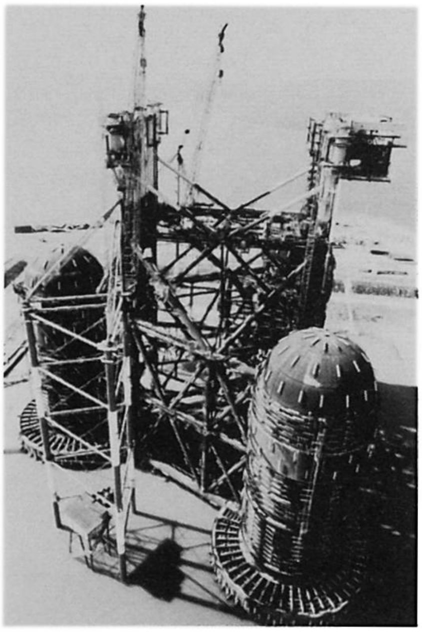


Fig. 6 A tall North-sea platform

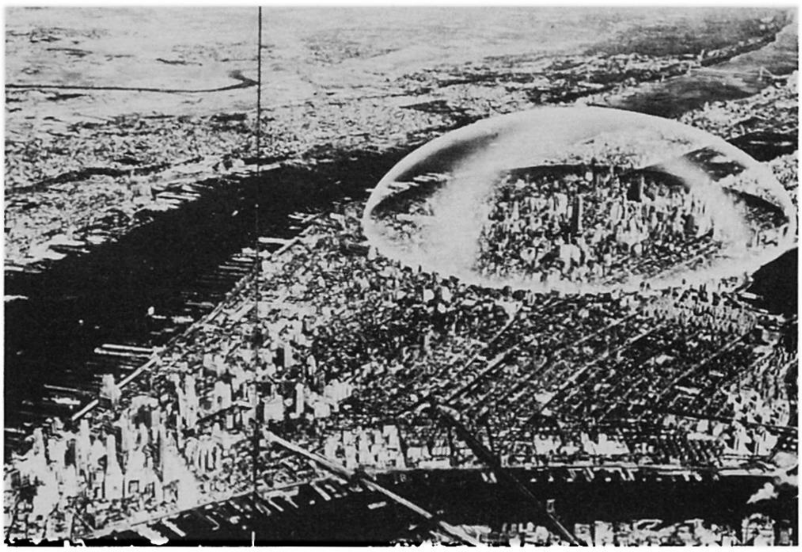


Fig. 7 A feasible plexiglass dome over midtown Manhattan

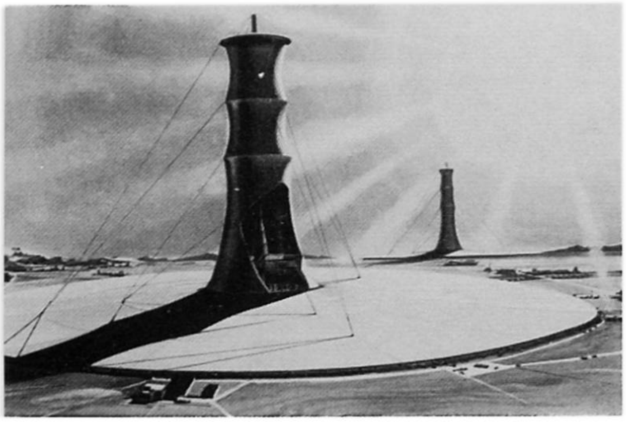


Fig. 8 Recovering solar energy

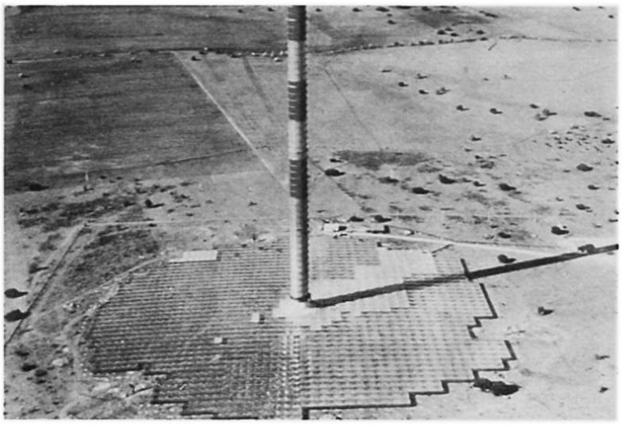


Fig. 9 Schleich's pilot power plant

airport installations, tanks for liquids and gas. But perhaps bridge construction, whether above or below the surface, offers the greatest challenge. In Europe alone of pressing importance, and calling for solution within this century, are the problems of crossing the Straits of Gibraltar, Messina (Fig. 10) |9| and Dover, while Japan has problems of at least equal importance for road and rail connections between its islands. But why should we be limited to this planet?

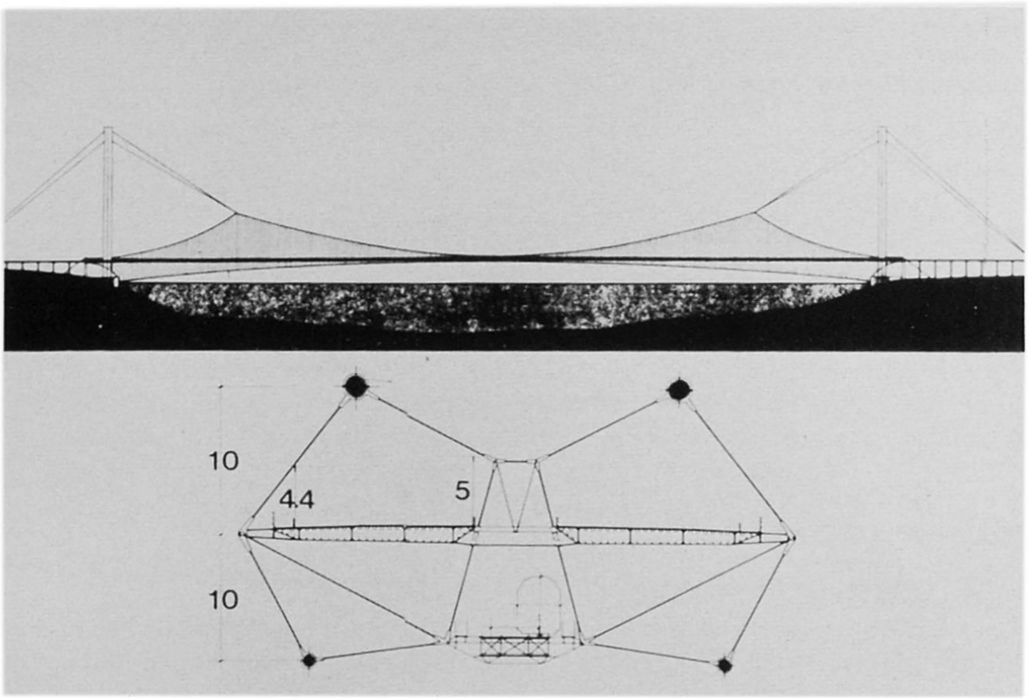
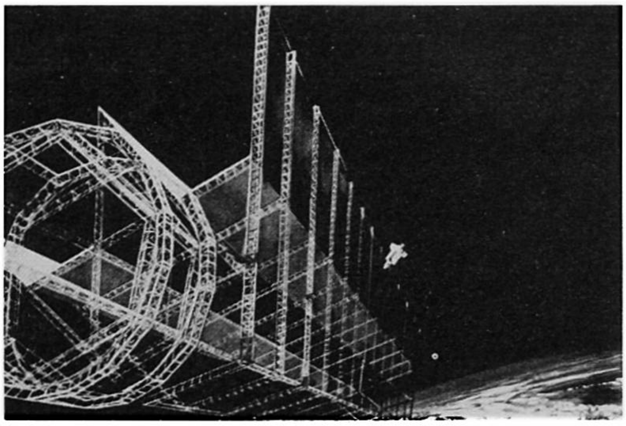
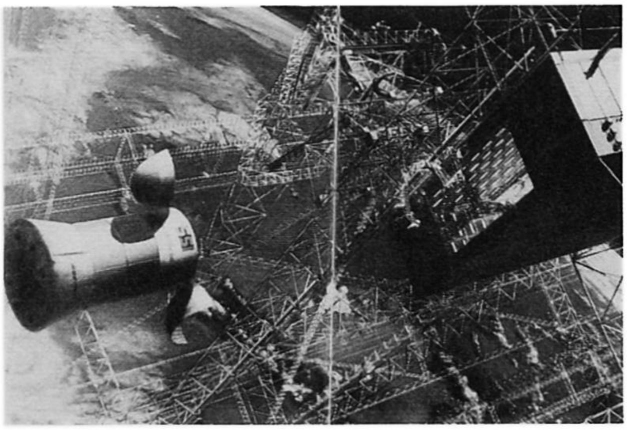


Fig. 10 "Hanging" bridge for the Messina Strait crossing

Theodor von Karman, when addressing a conference on scientific models at Venice in 1954, spoke about landing on the moon and planetary exploration by means of rocket propelled vehicles. Many in the audience doubted if this was suitable for a rigorously scientific conference. But less than thirty years later we already have to face the problem of building large artificial satellites to orbit around the earth (Figs. 11 and 12), access and construction being facilitated by vehicles like the Shuttle |10|.

This will really be a fresh field for the structural engineer, accustomed as he is to struggling with the forces of gravity, but perhaps less prepared to deal with the problems of temperature variation that here will assume a role of much greater importance.

Figs. 11 and 12 Space platforms

5. PHYSICAL, MATHEMATICAL AND NUMERICAL MODELS

New materials, new structures, new environmental situations will raise new design and checking problems for the structural engineer of the 1990's. Among the new tools that we already have, but that will be even more formidable in the near future, are the computers, rapidly becoming more available to small groups or even individuals, easier to handle and, of great importance for the structural engineer, suitable for displaying the input and output.

The same sort of thing can also be said for optimisation techniques, which are becoming more and more refined - another area of fundamental importance for improvements in design.

The physical model, on the other hand, referring to an entire structure - the sort of model that men like Esquillan, Nervi and Torroja used to try out their brilliant ideas - seems to have a less happy future. But the trend towards industrialized building tends to stress the systematic use of laboratory tests on constructional details, such as fasteners and structural nodes. All modern universities have suitable test-beds for trying out these structural elements in realistic conditions, applying pre-fixed loading histories.

6. FABRICATION TECHNIQUES

The modern building is more becoming an industrial product.

The building trade itself, one of the most backward in terms of industrialisation, is rapidly trying to make up for lost time. So structural engineering has to find solutions that are optimal also in the sense that they are suitable for mass production. The key-words today are unification and standardisation, i.e. the aim of research is to find satisfactory final solutions made up of a limited number of components that are suitable for mass production. This system, which we all used as children when we played with our "Meccano" sets, has already registered a number of notable successes. The Buckminster Fuller domes (Fig. 13) and the Mero space trusses (Fig. 14) are typical examples. But these are perhaps extreme cases. However, even when such a high degree of unification is not possible, the trend is anyway to increase as much as possible the quantity of factory work on the components with a consequent reduction in the amount to be done on site. When even this is not possible, and wherever the size of the construction permits, an actual workshop is first built on the site for the pre-assembly of the components to be erected. As a result, the necessary hoisting equipment is already impressive, and will be even more so in the future. Shipyards and the oil industry are already employing cranes with carrying capacities of 10 MN.

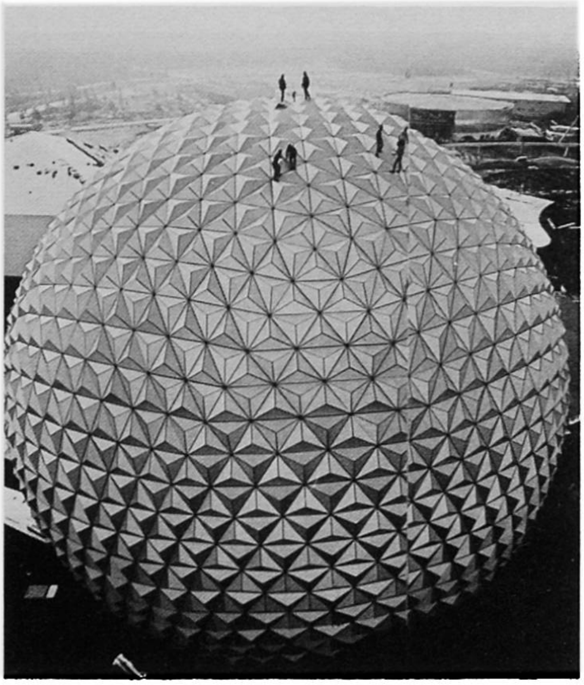


Fig. 13 Fuller type dome in Florida U.S.A.

Fig. 14 Space truss of the Mero system

Another aspect of this process is that not only the structures themselves are pre-assembled, but they often come fully equipped with all the necessary installations and finishings, so that once erected all that is required is a little fastening and sealing, and the building is ready for use.

7. ON SITE ERECTION TECHNIQUES

There have been many notable advances in the field of erection techniques, so that the right choice is a matter of considerable importance. When bidding for a bridge-building contract, for example, the erection technique may well be decisive. Nowadays, this generally means prefabricating the beams on the site and then sliding them forwards until they jut out, or else one proceeds by sections, cantilevered symmetrically over the piers (Fig. 15). The use of bentonite for diaphragm walls or large diameter piles has revolutionized the methods, times and

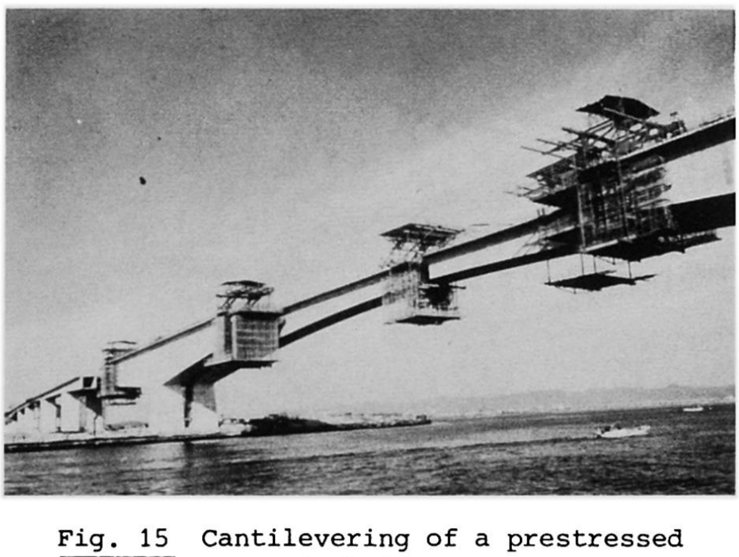


Fig. 15 Cantilevering of a prestressed concrete bridge

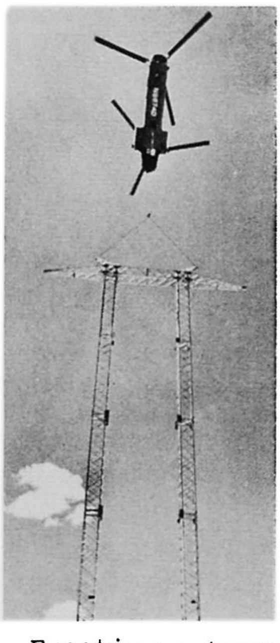


Fig. 16 Erecting a transmission tower by helicopter

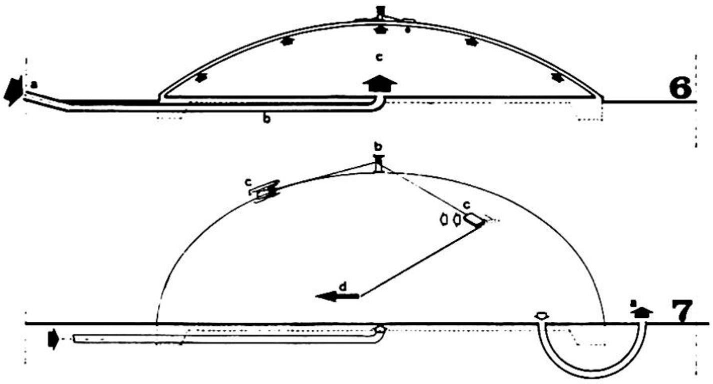


Fig. 17 Parashell inflatable formwork system

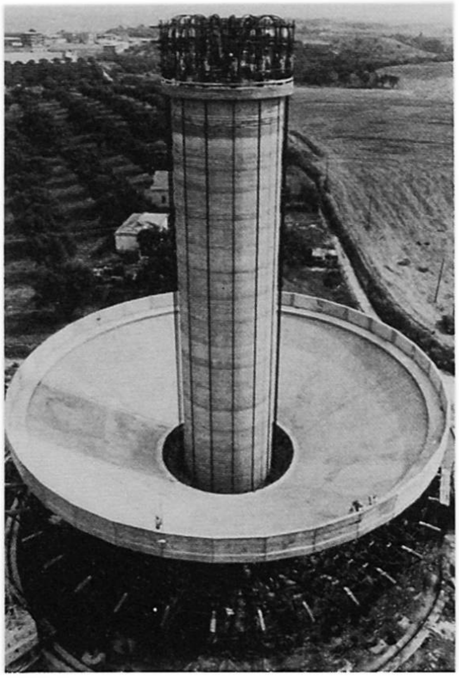


Fig. 18 Lifting an elevated reinforced concrete water tank

costs of foundations work. The use of vertical sliding or climbing formwork and of horizontal forms supported by joists as well as reinforced concrete "predalles" has eliminated or any-way greatly reduced the tubular scaffolding in building sites.

Derricks for erecting latticed towers have sometimes even been replaced by helicopters (Fig. 16).

The proper use of retarders makes it possible to cast a layer of reinforced concrete on a flat rubber sheet. Air is then blown under the sheet to inflate it, thus creating a dome of considerable size without the use of formwork (Fig. 17).

Finally, one might mention the hydraulic jacks that hoist entire packs of floor slabs or elevated tanks from their assembly at ground level to their required height (Fig. 18).

8. CONCLUSIONS

So far as structural engineering is concerned, the "new frontier" means that there are new goals to be reached. These goals, of course, will be the demands of an evolving society.

Some of them are already fairly clear.

The population explosion, especially in economically developing areas, and migration towards crowded conurbations, raise the basic problem of massive housing programs at low cost. There is no easy solution, and structural engineering has been playing its part for some time now. No great successes have so far been registered, but this has not been for lack of trying. Solutions must be found in the near future, and the structural engineer will be called on to give his contribution.

Housing, however, satisfies only one of man's basic needs. Social life also demands space both for work and for relaxation, and this implies areas to be covered and organised and inter-connected with residential districts.

There is also the demand for energy in all its available forms: heat, water-power, geothermics, nuclear, wind and solar energy.

New horizons are opening here too for the structural engineer. In fact, if one thinks of the offshore platforms, it may well be that the structural element is even more important here than for housing.

The ever-increasing mobility of modern man also raises fundamentally structural problems. New transport systems mean new bridges, viaducts, tunnels.

All these trends can easily be seen in present day society. But if we want to prepare for the future, it is not enough just to extrapolate from the present. We must bear in mind possible developments of situations that are now only in embryo but that may evolve as a reaction to the unfavourable consequences of present policies. From this point of view it seems quite probable that men will have to learn to live and work in areas that today would be considered hostile - deserts and other areas subject to extreme climatic conditions. It already seems probable that the sixth continent (the oceans) will be increasingly subject to the works of man, both above and below the surface. Man in space is already a scientific and technical fact, but not yet a social problem. The exploitation of the oceans, however, and of their immense energy resources, is already a question for the immediate future, and some interesting work has already been done (Fig. 19).

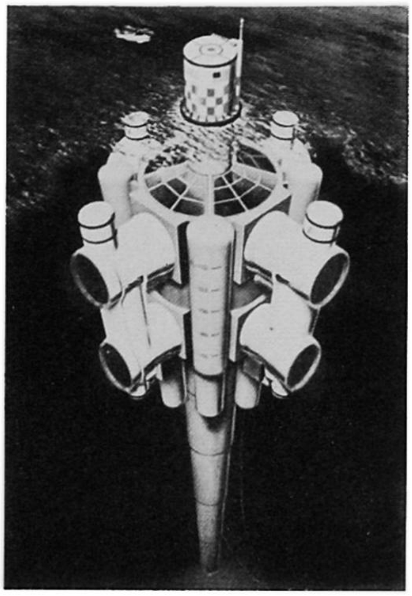


Fig. 19 T.Y. Lin Ocean thermal energy conversion (OTEC)

As to structural engineering, we can see the many problems that have yet to be solved, but really new ideas, by their very nature, have still to come into the open. They will come from architects, engineers, builders and even, why not, from outsiders. But it is not easy to foresee what they will be.

This introductory report is directed to those who have these new ideas, deriving from solid scientific and technical bases, to encourage them to bring out their new concepts, new methods, new techniques, to help structural engineering reach its new fontier.

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Seminar I

Health and Safety in Construction

Prévention des accidents dans la construction

Arbeitssicherheit im Bauwesen

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SUMMARY
„Health and Safety" has proven to be a far greater problem in construction than in any other industry. Accident statistics give some information only, and is merely used for insurance calculations. Unsafe situations occur far more often than we can discover from reported accidents and incidents. Frequency ratios from different countries are not comparable and often unreliable. Risk analysis could be a tool for health and safety management.

RESUME
Le thème „Santé et sécurité" est un problème bien plus important dans la construction que dans les autres branches de l'industrie. Les statistiques d'accidents ne donnent que certaines informations utilisées essentiellement dans des calculs d'assurance. Des situations dangereuses se produisent bien plus souvent qu'il n'est possible de le découvrir à partir de rapports d'accidents et d'incidents. Les fréquences et les valeurs statistiques ne peuvent pas être comparées d'un pays à l'autre et sont souvent sujettes à caution. L'analyse du risque pourrait être un outil intéressant pour la gestion de la santé et la sécurité.

ZUSAMMENFASSUNG
Das Thema „Gesundheit und Sicherheit" ist ein grösseres Problem im Bauwesen als in anderen Industriezweigen. Unfallstatistiken geben nur gewisse Informationen, welche vor allem für Versicherungszwecke gedacht sind. Gefährliche Situationen sind viel häufiger, als es die in den Statistiken erfassten Unfälle und andere Einwirkungen auf die Gesundheit aufzeigen. Die berichteten Häufigkeiten aus verschiedenen Ländern sind nicht vergleichbar und sind oft nicht zuverlässig. Risikoanalysen könnten ein Werkzeug für das Management der Arbeitssicherheit sein.

HEALTH AND SAFETY IN THE CONSTRUCTION INDUSTRY

1. INTRODUCTION

1.1 Tokyo, 1982

In August - September 1982 a workshop was held in Tokyo, organised by the Japanese Group of IABSE and by Working Commission IV "Construction Management", dealing with the subject "Health and Safety in Construction". During a Colloquium and a Symposium, experts of Asia, America and Europe listened, discussed and told about the developments in and the state of art of this subject.

We think it will be good to start this contribution to the introductory report of the coming IABSE-Congress, Vancouver 1984, with the final conclusions of our Japanese workshop:

1. In the Construction Industry we have a big problem in the field of Health and Safety
2. The number of accidents in the Construction Industry is relatively far more higher than in all other industries
3. Accidents come from unsafe situations; the unsafe situations occur much more often than we can read from our accidentratios
4. Accident-ratios and Health-ratios should be related to each other; an equal definition of these ratios in different countries would give a possibility to compare.
5. Looking to these ratio-figures is one thing: far more important is to find a feedback to prevention of accidents.
6. Safety-risk-analysis gives us a tool of management to more safe working conditions
7. Safety-planning and -programming is a necessity for each construction company and for each constructionsite.
8. Safety-measures should start at the sources of possible unsafe actions and circumstances.
9. Safety and Health should be subject for more research and developping programs.
10. Safety is the responsibility of all partners in the construction process.

1.2 Vancouver, 1984

Part of the IABSE-Congress 1984 will be a Symposium dealing with Health and Safety. During this symposium we want to have new contributions to this problem-field and we hope to get more information to give answers to questions which we will prescribe in the following paragraphs of this paper.

2. HEALTH AND SAFETY: A BIG PROBLEM

2.1 Accidents during construction

When we read the different safety-reports from various countries, we find the Construction Industry has a high rate of accidents, serious accidents and fatal accidents in comparison to other kind of industries. If we reckon with 'all industries', the Construction Industry has in most countries twice-time more serious (incl. fatal) accidents than we find to the total industry.
And, we have to mention that the published figures of different countries are often not correct: they all speak from 'reported accidents'. What happens in the field of 'not reported accidents' is unknown, and often a fatality after some months of sickness followed on an accident, is not counted as a fatal accident. Also we do not know anything about the figures of 'nearly-accidents'.
So, speaking of 'unsafe situations', we only have some speculations.

2.2 Safe and unsafe situations

Thinking about safety, we state here that there is no safe or unsafe situation in an absolute way. Accident prevention is only possible if we can foresee certain unsafe situations, unsafe working circumstances, unsafe actions of management or of the workers.

Can we weigh in one way or another, the chance of occurance of certain unsafe working conditions? And if so, will such a situation lead to an accident? Why yes or why no?
In what way or to what extent will count our own experiences: in what way are we remembered to unsafe situations, to possible accidents?
Can we imagine certain unsafe circumstances? How can we prevent those accidents, which never have happened before?

2.3 Backgrounds of accidents

Every accident will have a certain background, an environment in which through certain unsafe circumstances or unsafe acting it may occur.
When we define an accident as: 'a sudden default of an availability, caused by an unattended disturbance of the usual course of events, or of the fixed way of working', we can try to find out why such accidents will happen. The causes can be brought back to: wrong methods, wrong means, wrong actions, poor working climate, poor organization, wrong mentality, poor management.

But, when we speak about those accidents as a sudden default, we forget that through certain unsafe working conditions, our health can be destroyed by poison, radiation, noise, stress, ... These causes are not sudden actions, but very slow influences which bring damages in our body after several years.

So our problem field is much wider than looking after 'accidents': it includes unsafety in its totality. We see and recognize only the top of the 'ice-berg of unsafety'.

2.4 Factors of influence

Each worker on a building site works under certain circumstances which will be regulated by several factors. Some of these factors can be influenced by management; some of them are bound to the worker himself and cannot be directed by management. These latter factors are coming from his own environment, family conditions, social problems, etc. The first mentioned factors come from this special job, on that special site, constructed by this very contracting company.
The worker does his job under all these conditions, and suddenly there is that accident! Why, why now and why did it not happen before or to the other workers?

Under what kind of conditions, on what kind of work, on which site, in what organization, to which workers, come accidents to a realization?

3. SAFETY AND COSTS

3.1 Costs of accidents

In some countries a lot of attention is paid to the cost-consequences of accidents. We think it is good to be aware of the costs of accidents, to know how we can invest in prevention measures of those accidents and incidents.
However, we have the impression that costcalculations of accidents are mostly used for insurance-calculations and not for accident prevention in a direct way. In American literature we read about:
- Insurable costs

and:
- Not insurable costs.

The insurable costs vary in different countries also in different ways, due to different social legislation. The not insurable costs are the other costs which are to be paid by the employers or by the employees.
All costs are losses to the society, which could be prevented when no accidents should occur.

3.2 Costs of accidents-prevention

On the other hand we can count the costs of all the prevention measures, and we can imagine that the more prevention measures we take, the lesser accidents will occur.

3.3 The safety-costs-line: optimum costs

When we count the costs of safety-measures to the costs of accidents together, we find a curve which gives us the relation between safety and costs (fig. 1).

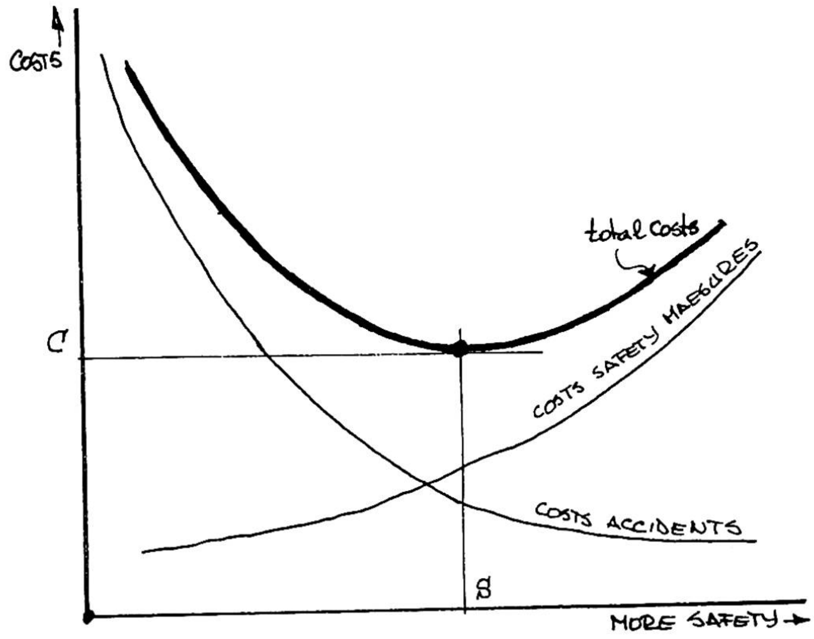

Fig. 1: Safety and Accident Costs

It is clear that we can imagine us one point in this curve in which we can speak about optimum costs. From an economic point of view, this point will bring the biggest profit to the employer.

3.4 The safety-costs-line: optimum safety

In Tokyo we discussed also the following question: (fig. 2)

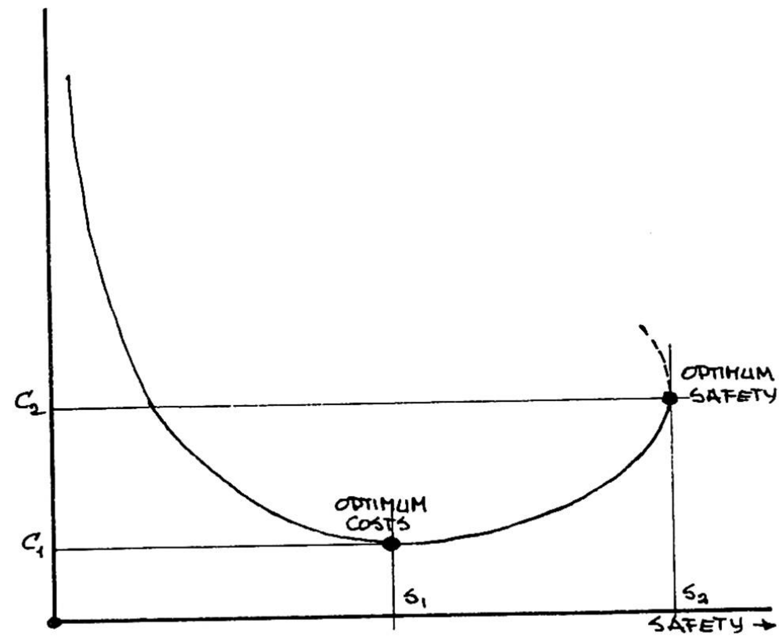

Fig. 2: Minimum Costs or Maximum Safety?

If we put in more safety-measures, it can be that the total costs will rise again, but that we have a safer construction-site. Is there one point where we can speak of optimum safety? And what happens after that? Putting in still more safety-measures, will give relatively more costs, but will it also give us more safety?
Some contractors think that too much safety measures, give us after such a point no more safe, even unsafer working circumstances (fig. 2).

Questions are:
- Do we recognize such statements?
- Have we any research-results on this subject?
- Should we think in terms of maximum-safety or in terms of minimum costs?

3.5 Safety-costs line on each construction site

When we think in such a safety-concept, we state that each construction site will have its own safety-costs-curve. The shape of this line will give an impression of the kind of work which will be constructed on that site. Every construction work will have his special problems in the way the work will be done, technical but also in the field of health and safety (fig. 3).

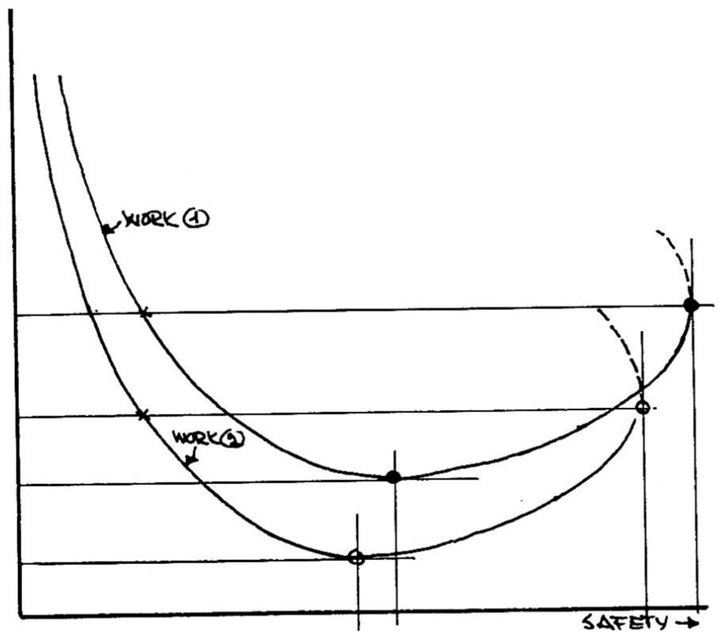

Fig. 3: Every Construction-site has its own safety-conditions

4. ACCIDENTS

4.1 Ratio, Statistics

In all countries, we try to find statistical data to get an information in this field of health and safety.
We try to think in terms of ratio, as:
- Accident-ratio
- Sickness-ratio.

In all countries these ratioslook like each-other, but they all differ in one way or another; it is impossible to compare them. The ratios are calculated along two directions:
- Accident Frequency (AF)
- Accident Heaviness (AH) or: Accident Severity.

To compare Accidents, with Health, it should be good to compare also:
- Sickness Frequency (SF)
- Sickness Heaviness (SH) or: Sickness Severity.

To compare the figures of different countries, we should count these ratios in the same way.

We suggest:

1. $$\text{A.F.} = \frac{\text{number of reported accidents}}{\text{number of men years}} \times 100\ (= \text{----}\%)$$

2. $$\text{A.H.} = \frac{\text{number of reported lost days}}{\text{total worker days/year}} \times 100\ (= \text{----}\%)$$

3. $$\text{S.F.} = \frac{\text{number of reported sick-cases}}{\text{number of men years}} \times 100\ (= \text{----}\%)$$

4. $$\text{S.H.} = \frac{\text{number of reported lost days}}{\text{number of worker days/year}} \times 100\ (= \text{----}\%)$$

When we act also, we can compare different countries if the data brought into the ratio are reliable and counted in the same way.
Therefore some questions are still there:
- What are accidents?
- What are reported accidents?
- When do we count the lost day? In my opinion that must be already the first day of absency and not the 4th or the 8th day! That means: often other data than we use in different countries due to our social securities.
- How are those data 'reported': how many are not 'reported'?
- We count the real days, and not some additional day-figures which depend on the severity of the accident itself: the loss of fingers, hand or eye should be registrated in another way.

If we handle on this way, we come to the following quotation for the Netherlands in round figures in the construction industry (approx.):

HEALTH $$\text{S.F.} = \frac{485{,}000}{350{,}000} \times 100 = 138.57\%$$

$$\text{S.H.} = \frac{9{,}650{,}000}{77{,}000{,}000} \times 100 = 12.53\%$$

SAFETY $$\text{A.F.} = \frac{18{,}000}{350{,}000} \times 100 = 5.14\%$$

$$\text{A.H.} = \frac{450{,}000}{77{,}000{,}000} \times 100 = 0.58\%$$

4.2 Causes of accidents

We mentioned already the influences from outside: initiated by social -, family -, wheather -, etc. conditions as one of the range of causes of accidents.

A second group of causes is: personal short-comings. To this group we reckon: not knowing, not capable to do the work, not wishing to do such work, wrong mentality, ...

The next group can be mentioned under the title of: unsafe actions, to which belong: unqualified activities, unsafe working place, safety devices put out of order, use of unadeqaute equipment, unsafe loading or unloading, unsafe working conditions, unsafe way of joining elements, unsafe working near -, on - and - with moving equipment, disturbances in the work, not using personal protection.

The following group is brought together under: unsafe situations. With this group we enter the field of management: poor working organization, insufficient protection, unsafe working sites, unsafe use of equipment, unsafe ventilation, vibrations, noise, unsafe clothes, inadequate personal protection, wrong mentality.

4.3 Accident-effects

All countries try to give an impression of the effects of accidents to human beings. We shall try to give some output-data from several countries.

4.3.1 Japan:

In 1978 there were 118.568 reported injured workers: fig. 4:

injured workers	1978	
	total	±%
civil work	45,546	38
building work	64,086	54
equipment work	8,936	9
total	118,568	100
deaths	1,583	1.75

Fig. 4: Injured workers Japan (1978)

In 1981 Japan counted 1173 fatal accidents: fig. 5:

causes fatal acc.	total	%
falling	425	36.2
break-down	91	7.8
collapse	97	8.3
machinery	425	36.2
electricity	48	4.1
fire, explosion	18	1.5
handling	10	0.9
others	59	5.0
total	1,173	100

Fig. 5: Fatal accidents Japan (1981)

4.3.2 England

In 1978/79/80, England reported the following list of causes of accidents and of injured people (fig. 6):

causes and injured people	1980		1979		1978	
	total	fatal	total	fatal	total	fatal
1. stepping in striking against subjects	2279	-	2487	-	2723	1
2. collision with materials	151	8	194	10	257	9
3. working with tools	2434	-	2112	1	2323	-
4. working with equipment	2967	28	3154	33	4314	27
5. falling heights	4401	65	4663	55	5044	56
6. falling flat	3933	2	4357	1	4198	-
7. falling materials	1652	10	1829	10	2368	14
8. other accidents	3190	6	3221	-	3861	3
9. occupational diseases	178	7	175	9	183	10
10. unknown	8305	1	8814	-	7714	-
Total	29,490	127	31,006	119	32,980	120

Fig. 6: Causes of accidents England ('78/'79/'80)

4.3.3 The Netherlands

For The Netherlands we find the following figures (fig. 7):

	causes and injured people	1980		1979	
		total	fatal	total	fatal
1.	stepping in nails	886	0	846	0
2.	materials	6969	6	8659	5
3.	tools	3083	0	2764	2
4.	equipment	955	4	479	3
5.	falling heights	262	5	122	2
6.	falling flat	2381	1	3001	4
7.	falling from ladders	827	2	772	1
8.	other accidents	1527	11	285	3
9.	occupational diseases	114	0	127	0
10.	unknown	35	1	14	0
	total	17,039	30	17,066	20

Fig. 7: Causes of accidents The Netherlands ('79/'80)

4.3.4 Germany

In Germany we find figures of the building industry in comparison to the total industry. We give a quatation of accidents, of serious accidents and of fatal accidents in fig. 8.
(In Germany an accident is counted as every incident which causes an absence of men for more than three days).

year	menyears building industry in % of total ind	accidents to 10.000 my		serious accidents		fatal accidents	
		build ind	total ind	build ind	% total ind	build ind	% total ind
1970	10.7	154.81	99.63	86.66	16.9	51.0	17.0
1971	10.7	156.99	95.23	94.80	18.4	59.7	20.0
1972	10.5	151.28	88.23	93.64	19.0	51.5	19.0
1973	10.4	139.42	85.16	93.33	19.7	54.5	20.3
1974	9.9	122.88	76.35	89.23	19.3	46.7	19.1
1975	9.3	114.80	68.14	79.78	19.0	42.5	20.5
1976	9.6	121.87	73.37	76.36	19.4	43.2	20.6
1977	9.5	118.76	72.15	78.50	19.6	39.6	19.9
1978	9.4	120.70	70.78	74.81	19.3	35.0	18.2
1979	9.7	119.85	72.56	75.10	18.4	39.2	19.6
1980	9.7	120.92	71.69	76.31	19.1	36.9	20.4

Fig. 8: Figures of Germany

4.4 Some considarations

If we look to the different way of data-gathering, we think we have to ask ourselves to what purpose we want to collect them. If we take safety as an starting point we want to collect such an information which will give us the background of:

- Causes of accidents (falling, striking, ...)
- Seriousness of accidents (part body, fatal)
- Amount of accidents (total, proportional)

The information should give us all accidents and the accidents which caused injuries to people: inside on the working place and outside by passing people, neighbours, not workers.

The in par. 4.1 suggested ratio-figures will give us some information, with which we could compare different countries and different industries.

Figures of accidents to every 10.000 manyear will also give us some clear information, when also the first day of absency is counted.

The combination of causes and of damaged parts of the body, could give us information to better ways of protection and of those parts of the building proces that should be changed to make it possible to work in a safer way.

As an example we give for the Netherlands the year 1980 (fig. 9):

Causes	1	2	3	4	5	6	7	8	9	10	TOTAL
Part of the body:											
1. Head	0	612	76	80	31	90	46	124	0	1	1060
2. Eyes	0	811	135	14	2	11	7	90	0	7	1077
3. Hand(s)	0	2854	2342	460	24	353	140	479	77	8	6737
4. Foot	886	1471	299	227	56	1024	258	220	0	8	4449
5. Inside	0	635	106	123	117	649	217	232	37	5	2121
6. Other	0	586	125	51	32	254	159	382	0	6	1595
Total	886	6969	3083	955	262	2381	827	1527	114	35	17039
Time of absence:											
0 - 1 week	497	1971	844	186	27	498	150	347	1	12	4533
1 - 3 weeks	380	4080	1934	585	134	1471	458	823	8	20	9893
3 - 6 weeks	5	591	204	114	34	218	122	240	49	0	1577
6 -13 weeks	3	195	56	38	27	105	46	62	30	2	564
13 - more weeks	1	126	45	28	35	88	49	44	26	0	442
Hospital	7	325	145	88	70	175	78	151	0	2	1042
Fatal	0	6	0	4	5	1	2	11	442	1	30

1 = stepping in nails
2 = materials
3 = tools
4 = equipment
5 = falling heights
6 = falling flat
7 = falling ladders
8 = other accidents
9 = occ. diseases
10 = unknown

Fig. 9: Figures of The Netherlands ('80)

5. THE SAFETY PLAN

If we really want to come to safer working conditions in our construction industry, we have to work with a safety plan.

Such a safety-plan must be supported by the board of the company and the topmanagement of the organization, it should be brought into action and be advised by the safety-department, when necessary advised by an external adviser.
The safety must be known by every member of the organization, brought into action on each site and introduced to every newcomer at the start of each work.

The safety is based on the following considerations:
- Reduce human suffering
- Reduce loss of materials
- Promote morale
- Promote productivity
- Reduce insurances-rates
- Reduce costs.

And such a program should cover:
- The purpose of the plan itself
- The scope of that plan
- The responsibilities to management and workers
- The establishment of a safety committee
- The safety- and toolbox-meetings
- The measures for personal protection.
- The instructions to be given
- The organization of safety-publications
- The different plans for external assistance
- The measurements for care and transportation of injured people
- The investigation of accidents and unsafe situations
- The incident-and accident reports
- The feedback to the organizations.

Thinking along those lines we come to a safety-decision-sceme; when we look now to the construction work on our sites, we recognize the human factors and the material factors. We both should analyse them, to weigh certain chances. Is the risk acceptable or not? If yes, we do the work in the way it is foreseen and prescribed to realise. If we foresee an an onacceptable risk, we have to decide to do it in another way.

But, even when we take the calculated risk, there could happen something we do not want, or something that is not acceptable but what was not foreseen as such! In the working situation an accident could happen now or it doesn't happen. In both situations we have at this moment unsafe working conditions.
Only in the case of a real accident we meet damages and injuries.
Thinking in a better way along safety-decision-trees could bring our industry to a higher level of our safety-performance.

Seminar II

Computer Aided Structural Engineering

Génie des structures assisté par ordinateur

Computergestützter konstruktiver Ingenieurbau

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Steven J. Fenves, born in 1931, received his degrees in civil enineering from the University of Illinois, where he taught until 1972. His teaching and research activities deal with computer-aided engineering, with emphasis on representation of standards, databases and expert systems.

SUMMARY
Computers and computer-based methods have already significantly affected the practice of structural engineering. New developments in computers, computer graphics, databases, expert systems and intelligent construction equipment promise even bigger changes. To benefit fully from these developments, existing disincentives have to be removed.

RESUME
L'ordinateur et les méthodes basées sur l'ordinateur ont déjà influencé profondément la pratique du génie civil. De nouveaux développements dans le matériel, l'infographique, les systèmes de bases de données et le contrôle numérique de l'équipement de construction promettent encore de plus grands changements. Pour bénéficier pleinement de ces développements, il est nécessaire de faire disparaître tous les éléments décourageants encore existants.

ZUSAMMENFASSUNG
Computer und Computermethoden haben die Praxis des konstruktiven Ingenieurbaus schon tief beeinflusst. Neuere Entwicklungen in der Computer-Hardware, in der Computer-Graphik, in der Datenbanktechnik und in den digital gesteuerten Konstruktionsverfahren versprechen sogar noch grössere Einflüsse auszuüben. Um von diesen Entwicklungen ganz zu profitieren, ist es notwendig, alle noch vorhandenen entmutigenden Faktoren aus dem Wege zu räumen.

1 Introduction

Over the past 25 years, computers have taken on an increasing role in structural engineering practice and research. Computer programs have been developed to assist in every phase of structural design, analysis and construction. Yet we are only a short distance into the "computer revolution". The emergence of powerful personal computers, vastly expanded computer graphics, widely accessible distributed databases, microprocessor-controlled "intelligent" construction equipment (soon to be augmented by a wide range of construction robots), and knowledge-based expert systems will all drastically change structural engineering design and construction practices, and even the nature of the structures we design, build and operate. In order to take full advantage of these developments, the structural engineering profession must remove many of the existing disincentives due to the professional, organizational and regional dispersion of the profession.

The purpose of this general report is to provide a focus for the Seminar and Poster Session on Computer-Aided Structural Engineering. To set the scene, the processes of the structural engineering profession are modelled as a four-level nested hierarchy of "programs" in Section 2. The potential contributions of informatics, incorporating computer-aided and computer-based methods in the broadest sense, are presented in Section 3. The present status of computer-aided structural engineering is summarized in Section 4. Section 5 present a tentative list of further potentials, while Section 6 deals with some of the barriers and disincentives to overcome. A brief summary and conclusion is given in Section 7.

2 The Structural Engineering Process

In a general report dealing with computer-aided structural engineering, it is appropriate to model the structural engineering process by computer "programs".

At the innermost level, the activity of a design organization designing a structure may be represented by the procedure Design.

```
PROCEDURE Design;

   BEGIN

      {input:  standards, design specifications;
               client needs, program, constraints;
               knowledge of construction practices;
```

```
            office design experience}
       conceptual design {synthesize structural configuration};
       analysis {predict response};
       detailed design {proportion components};
       evaluate constraints;

       IF design unsatisfactory THEN

            REPEAT

            modify structural parameters;
            redesign;
            re-evaluate constraints

       UNTIL design satisfactory;

       produce design documents;
       record/modify design experience;

  END design.
```

The extend of redesign and re-evaluation is highly variable, and may range from a full iteration starting from a new conceptual design to minor parameter adjustments.

At the next level, the activity of a design-build organization is represented repeated by the procedure Design-build.

PROCEDURE Design-build;

```
  BEGIN
       {input:  standards, design specifications;
                client needs, program constraints;
                construction experience}

       Design;
       evaluate buildability;

       IF design not buildable THEN

          REPEAT
               modify construction practice knowledge;
               redesign;
               re-evaluate buildability
          UNTIL design buildable;

       build;
       record/modify construction experience;

  END design-build.
```

It is to be noted that if design and construction are contractually separated (as in public bids based on completed designs), much of the feedback indicated by the model cannot take place. The activity of a major owner is modeled by the procedure Commission-Operate.

```
PROCEDURE Commission-operate;
   BEGIN

      {input:  standards, design specifications;
               operating performance experience}
      formulate needs, program, constraints;
      Design-build;
      evaluate operating performance;

      IF building not operable THEN

         REPEAT
              modify building;
              re-evaluate operating experience
         UNTIL building operable;

      record/modify performance experience;

   END Commission-operate.
```

The overall process of the profession as a whole is represented by the program Structural Engineering Profession.

```
PROGRAM Structural engineering profession;
   BEGIN

      {input:  collective experience represented by standards
               and design specifications}
      Commission-design-build-operate;
      evaluate performance;

      IF performance inadequate THEN

         REPEAT
              modify standards;
              re-evaluate performance of buildings
         UNTIL performance adequate;

      record/modify standards

   END profession.
```

The salient points of these models are:

- there are multiple iterations at each level - the amount of iteration can be substantially reduced if the inputs are correct and are fully understood;

- there are two "outputs" at each level: the tangible "deliverables" (the design documents, the completed building) and the intangible increment of knowledge or experience, which provide the feedback that influences future activities; and

- at the outermost level, standards and design specifications represent the "collective memory" of the profession as a whole, in terms of empirical evidence that the requirements, methods and practices incorporated in the standards produce safe and serviceable structures.

3 Potential Contributions of Informatics

Computers and computer-based techniques can contribute significantly to the improvement and expansion of the processes sketched in the preceding section. The contributions can be grouped into four major categories.

Procedures. Undoubtedly the most common contribution is in the development of procedures, implemented as computer programs, for the many aspects of design, analysis and management. Programs of various levels of completeness and generality have been written for essentially every phase and aspect of structural engineering. The development of these programs has a two-fold benefit: the practical one of providing a computational tool, often for tasks and levels of modeling prohibitively expensive for manual processing, as well as the intellectual one of forcing the program developer to explicitly and critically examine and evaluate the procedures, limitations and assumptions used in manual processes.

Up to the present, all procedures implemented as computer program has to be algorithmic, implying that the program produces a unique and correct solution for every possible combination of conditions within its scope. As will be discussed in Section 5, the recent development of expert systems based on artificial intelligence methods provides a way to represent and process heuristic knowledge, consisting of the empirical knowledge and "rules of thumb" which characterize much of structural engineering expertise.

Interfaces. The growth in complexity of programs, and the desire to integrate programs initially developed for separate tasks, both contribute to the attention being paid to interfaces between programs and their users. Man-machine interfaces, in the form of "user-friendly" programs and particularly computer graphics, contribute significantly to raising the man-machine dialog to the level of the engineer, and to the visualization, understanding and "internalization" of the complex phenomena manipulated by the programs.

At the same time, the need to interface separate programs provides the impetus for the development of design databases, which can serve as the active repository of the highly dynamic data that emerge in the design process. Thinking about and attempting to structure this collection of information has the same intellectual benefit as the process of procedure development.

System Concepts. The increased integration of procedures, programs and data naturally leads to a system view of structures, with the goals, environment and hierarchical constraints among components and their responses defined much more explicitly than in the past. Equally important, these concepts lead to viewing the design process itself as an operational system, with the individual activities coordinated and managed in a consistent fashion. This systems viewpoint permits considerably tighter integration in breadth (among the participating design professionals) and in depth (across the design, construction, regulatory approval operations and management phases).

Sensors and Controls. The sensors used throughout structural engineering for data collection, for performance and environment monitoring and for fabrication and construction control are becoming increasingly sophisticated, and many of them now produce "on-line" digital signals that can be directly integrated with analysis and control processes. Similarly, fabrication and construction equipment is increasingly digitally controlled, and can accept their control information directly from the output of design programs.

4 Present Status

The following subsections summarize the perception of the present status of computer-aided structural engineering.

The Computing Environment. In computer hardware and access mechanisms, it is clear that the trend is increasingly towards powerful personal computers, providing substantial local processing capability for the individual engineer, but networkable to access special resources, such as large databases, large processors for occasional big computing jobs, plotters, etc. Software engineering tools and methodologies, initially developed for large programming projects, are being adapted to the more distributed environment of structural engineering practice. A variety of robust software components, including computer graphics, geometric modeling, database management systems, word- and text-processing are increasingly being integrated into structural engineering software and systems. Commercial CAD systems are also finding increased use. The first generation stations were purely drafting tools, requiring digitized or other manual graphic input, and were intended only to produce plotted output. These systems are being rapidly extended by "downstream integration" to produce bills of materials, parts lists and other derived information. Increasingly, these systems are also undergoing "upstream integration," so as to receive some or all of their data from preceding design operations.

The Professional Environment. There are some disturbing indications that computers are adding directly to the pressures of practicing professionals. Inclusion fo computer capability evaluation in the selection of consultants, contract requirements to use specific programs or systems, and insistence on refinements and tolerances achievable only by computing may be justifiable in specific instances, but their indiscriminate application by clients or regulatory agencies can be counter-productive and can restrict the range of the engineer's professional responsibilities.

Second, there is an increased disparity between analysis and design. Curiously, this phenomenon has two different manifestations.

For relatively simple structures, primarily framed structures, it is now common practice to produce a fully stressed design, i.e., iterate a few times on analysis and proportioning until every member is at the maximum allowable limit (stress, strength, deflection or other appropriate specified constraint) in at least one loading condition. We tend to forget that the limits embodied in our specifications and standards have been historically "calibrated" in a manual design environment where reanalysis was prohibitively expensive, and the design was considered satisfactory when a few key members were at their allowable limit in one loading condition. In the terminology of reliability-based design, the analysis error and its variance have been drastically reduced. Yet, we have not seriously questioned the impact of this development on the central safety factor.

In relatively complex structures, such as tanks and pressure vessels, the opposite occurs. The detailed computer model - practically always a finite element model - is so time consuming to construct and interpret that analysis is used primarily as a post-facto evaluation tool of design decisions previously made. In effect, for these structures, detailed analysis has largely been removed from the design cycle. This trend has been aggravated by the ease with which geometric design can be performed on CAD systems; it appears to have been migrated, but not eliminated, by the closer coupling to Design Analysis supported by the newer CAD systems.

Furthermore, the ready availability of complex analysis tools, especially those for nonlinear conditions, have presented many structural engineers with models and solutions which they cannot adequately comprehend. There is a general lack of guidelines and comparisons for using these advanced analysis tools.

As a counterpoint, many positive effects can also be identified. Certainly, new applications, new models and new methods continue to proliferate, and some are gaining increased usage.

A very significant positive factor has been the emergence of a healthy civil engineering software industry. Every issue of Civil Engineering, ASCE News and Engineering News-Record carries columns of ads for civil engineering software products. Hardware vendors and service bureaus are providing an increased range of civil engineering software produced and maintained by independent developers.

There is a genuine interest across the profession for closer integration of design processes both in breadth and depth. The technical feasibility of such integration is vastly improved by the availability of the appropriate support facilities, primarily those for database management and geometric modeling.

There is equal professional interest and concern for redressing some of the unbalances discussed above. This is evidenced by the increased interest in synthesis and optimization, intended to provide computer aids to the early stages of design, and in updating of standards and specifications so as to bring them more in line with computer-based techniques.

Finally, there is some renewed and broadened interest in cooperation and information sharing among computer users. In the US, NICE is incorporated as a non-profit organization. In other countries, notably Great Britain, Holland, Japan and Australia, there are much more active civil engineering users' groups, undertaking on a cooperative basis a number of research, educational and development activities. An international "umbrella" organization, FACE, is emerging as the top node of this network of cooperative activities.

5 Potentials

The new computer revolution promises to have a wide-ranging impact on the future of structural engineering practice and research. The following sections outline some of the expected developments. It is hoped that the papers in this Seminar and Poster Session will address in more detail some of these areas.

Practice. It seems clear that the personal computer will become the dominant mode of computer use in practice as well as in education. The personal computer of the 1990's will cost no more than its 1980 precursor, but will have 10 times the storage and 100 times the speed. It will also be flexibly linked to other processors, databases and output devices.

Graphics and CAD will be vastly expanded, supporting and augmenting the entire range of design activities. Designers will be able to visualize and "feel" the effect of all design decisions, and will be able to control graphically most aspects of the design process.

Integration of design activities through shared databases will become common, even among multiple organizations involved in a design project. The advantages of this approach will become so self-evident that major institutional and organizational changes will result. In particular, the traditional design-construction separation will begin to disappear; in a few years all prospective bidders will have access to the design databases in "machine-processable" form.

Finally, a much wider and more viable civil engineering software marketplace will develop.

Research. The mathematical modeling of physical phenomena will continue to be the "mainstream" of computer-based structural engineering research. There is pressing need to develop models of increasingly complex phenomena, such as fracture, non-linear problems of a great variety, coupled phenomena between the structure and its environment, and many others.

Closely allied to model development is research to provide experimental validation of analytic models. At the present, our analytical modeling capabilities exceed our understanding of materials and real structural behavior, especially in the inelastic range. This unbalance must be redressed, eventually leading to a new generation of reliable analytical simulation capabilities.

In parallel to the above research streams, deeply rooted in the traditional engineering research tradition, there will be increased research activity in more specific computer-related issues. Among these are: exploration of novel computer architectures, such as highly parallel processors, for modeling engineering problems; research on the role of graphics, geometrical modeling and design databases; and research on user interfaces for engineering users and engineer application developers. The profession's increased dependence on these tools have by now demonstrated to the academic community that research in these areas contributes as much, if not more, to the growth of the profession as traditional research in modeling.

Promising Areas. There are at least three computer-related areas, presently in their infancy in structural engineering, which promise to have an explosive growth in the next decade.

First, structural engineering, dealing with relatively static objects, has been barely touched by the <u>microprocessor</u>, which is already supplying large amounts of distributed intelligence and processing capabilities in many other civil engineering areas (e.g., traffic control, building environmental control, manufacturing, etc.). It is not yet clear how this inexpensive, sensor-based distributed computing capability will manifest itself in structural engineering, but it is bound to have a major impact, at least in new construction tools and methods, which will then affect design options and methods. Beyond that, the possibility of self-diagnosing buildings and bridges is conceivable, and eventually even dynamic control of structural response.

The second promising area is that of <u>expert systems</u>, computer programs which perform intelligent tasks currently performed by highly skilled persons. This area has recently emerged from computer science research in artificial intelligence to the point where it is becoming practical to think about expert systems in structural engineering, incorporating the heuristic knowledge acquired by experts through experience. Most of the practicing structural engineer's mental processes are not algorithmic, but heuristic. Potential applications of expert systems range from generative processes such as synthesis and preliminary design to interpretive processes such as design evaluation or failure diagnosis.

Distributed computing, sensing, control and expert systems combine with mechanical devices to produce <u>robotics</u>. This is another rapidly growing area which has not yet affected structural engineering. Certainly, today's robots do not have either the requisite mobility to "navigate" around a changing construction site or the versatility to handle the large number of "one-of-a-kind" components that comprise most structures. Yet robotics will clearly affect the way structures are built, by providing a way to reduce costs, increase construction safety and extend the construction "workplace" into hostile or dangerous environments. It is time to begin thinking about the problems and opportunities that will arise. In particular, thought must be given to the radically different structural schemes made possible by robotic construction.

6 Disincentives to Overcome

The realization of the full potentials of computer-aided structural engineering will require some serious thought by the profession as a whole, its clients, partners and regulators to remove some of the present disincentives. The major impediments seem to fall into three categories.

Program Development and Sharing. The diversity and dispersion of the structural engineering profession has so far prevented us from sharing program development effort and costs in a meaningful fashion. Admittedly, this is a difficult task which presents an interesting paradox. In software engineering, the value of utility programs is measured by their portability, that is, how easily they can be transplanted to a new environment. In structural engineering the measure of the design program's value is just the reverse: if the program truly reflects the design style of the originating organization, it is bound to fail when used by another organization, in that it may yield results in variance with that organization's assumptions and design style. Nevertheless, the profession, the emerging software industry and the various cooperative user groups must attempt to develop mechanisms to improve the program development and distribution process.

Project Information Sharing and Feedback. As illustrated in the first two procedures in Section 2, large volumes of information about the emerging design, as well as about its suitability and buildability, circulate among the design and construction organizations cooperating on a project. The horizontal and vertical divisions among the participating organizations often impede the natural flow of up-to-date information and the necessary feedback. As a minimum, standards for format and contents of "machine-processable" design and construction data are needed. Beyond that, computers and information processing techniques are needed to communicate back to designers the results of construction experience so as to guide future designs. A similar information flow needs to be established at the next outer level, to provide feedback on the operability and maintainability of completed structures.

Professional Information Sharing and Standards. Moving to the outermost "program" discussed in Section 2, mechanisms must be found to record and make accessible "on-line" information about past building performance, to serve designers directly as well as to provide the basis for the development of improved standards and design specifications.

7 Conclusions

Computers and informatics have already profoundly affected structural engineering, to the point where the qualifier "Computer-Aided" in the title of this Symposium is almost redundant. The foreseeable new developments in computers, information processing technologies, monitoring and control equipment, and in robotic construction methods will produce even more radical changes. It is hoped that this symposium will clarify the issues involved and provide a glimpse on an exciting future.

Seminar III

Transit Guideway Structures

Structures des moyens de transport en site propre

Tragwerke für Verkehrsmittel auf Eigentrassee

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SUMMARY
Evolution of transit technology and effects on design of transit structures are reviewed. Development of divergent technologies is described, using existing transit systems to illustrate differences in structural support requirements. Cost, aesthetic, and social challenges to structural engineering profession are discussed.

RESUME
Le rapport passe en revue le développement des moyens de transport en site propre ainsi que ses conséquences sur le projet de l'infrastructure. L'évolution des nouvelles technologies est présentée à partir de systèmes de transport existants. Les exigences nouvelles posées aux structures porteuses sont ainsi mises en lumière. L'article traite aussi des nouveaux problèmes posés à l'ingénieur civil, tels que coûts, esthétique, problèmes sociaux.

ZUSAMMENFASSUNG
Die Entwicklung öffentlicher Verkehrsmittel auf Eigentrassee und deren Auswirkungen auf die Gestaltung der Bauwerke werden besprochen. Die Entwicklung neuerer Technologien wird aufgrund bestehender Transportsysteme beschrieben. Dabei werden neuere Anforderungen an Tragwerke erläutert. Fragen der Kosten, der Aesthetik und der sozialen Probleme werden dem Bauingenieur neu gestellt.

1. INTRODUCTION

Structures designed for transportation represent some of mankind's finest achievements. Centuries of refinement have brought us from the primitive footbridge to today's sophisticated highway and railway bridge. Transit structures are an offshoot of this evolutionary bridge development. Unique for their controlled access, the definition of loads they sustain, and the high degree of member repetition, transit structures offer the engineer an opportunity to optimize structural design and to introduce innovative concepts to the design and construction practice.

The paramount objective of the urban transit system is to move people efficiently. The system and its structures must be cost effective and must meet often-stringent community architectural standards. Since the transit structure is a continuous link which ties diverse portions of the community together, the successful designer must be familiar not only with current transit technology and physical site restraints but also with the fabric of the system's urban setting.

Until recently, the architectural profession has had little input to the aesthetic design of the transit structure. Thus, the responsibility for appearance and acceptance of the structure within the community falls to the engineer. Within the engineering profession, it is primarily the structural engineer who has influenced the growth and direction of new transit development.

The twentieth century has seen a marked change in the evolution of transit technology and a corresponding change in the development of the structural systems needed to support transit systems. The extremely high cost of tunnel construction has called for greater emphasis on elevated transit solutions. This paper will examine several of the unique structural solutions developed to meet transit needs.

2. HISTORICAL DEVELOPMENT

The first few decades of this century saw the adaptation of railroad technology to several transit systems. Early examples in the United States include the Chicago El and portions of elevated transit systems in New York and Philadelphia (Figure 1). In Europe, the Wuppertal monorail, spanning the Wupper River, represents an original solution to the needs of elevated transit in confined urban spaces.

All of these early structures are designed and fabricated from built-up steel members. The influence of low labor costs and high material costs is evident in these structures when the change in cross-section depth at each span is examined. Considered archaic by current architectural and urban integration standards, these structures were considered engineering advances in their day.

The evolutionary changes in the past two decades have had a pronounced effect on the design of transit structures. Developments which have most affected structural design considerations are

- Development of precast concrete structural systems
- Placement of continuously welded rails directly on elevated structures
- Development of rubber tire transit systems

Secondary features affecting the design of transit structures include

- Increase in operating velocity of interurban trains
- Automatic train control

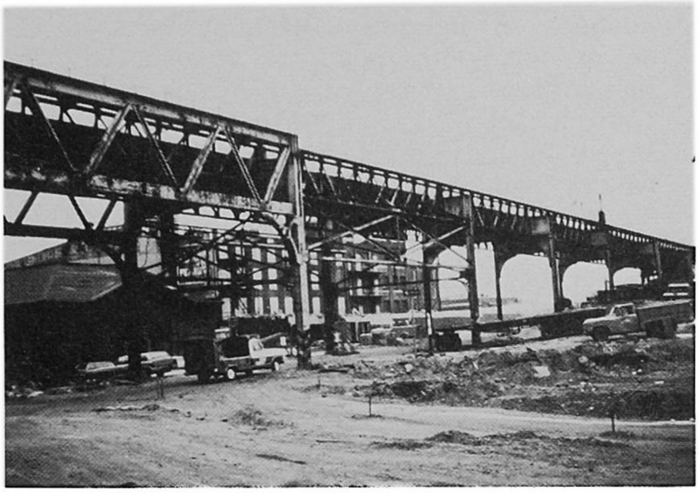

Figure 1 The built-up steel members of the Philadelphia transit system are indicative of early transit development. (Photo by author)

Figure 2 The Westinghouse Electric Corporation Skybus Guideway is a composite steel and concrete structure which uses a center steel rail for vehicle guidance and retention. This amusement park operation needs no walkways for emergency egress. (Photo by author)

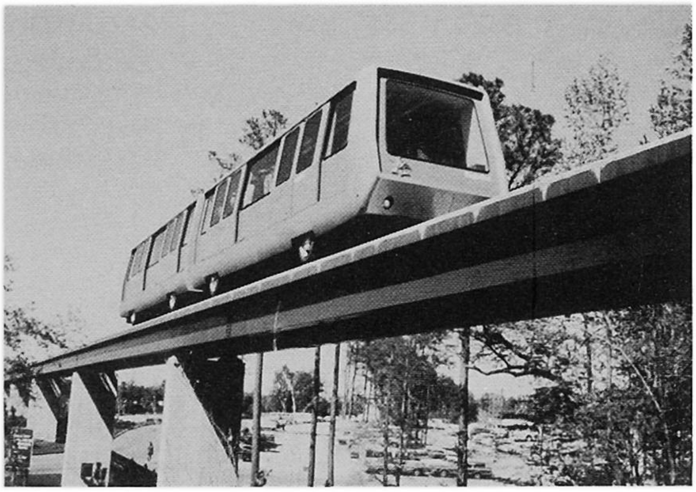

Figure 3 The Dallas-Fort Worth Airport uses a precast concrete box girder with side walls to support the transit vehicle. The vehicle steers off the top of the concrete wall. (Photo by author)

Most recently, the emergence of exotic suspension and propulsion systems is influencing the direction of structural design. Linear induction motors (LIMs), magnetic levitation, and air-cushioned suspension systems offer unique opportunities for new structural concepts, introduction of new materials, and development of new construction techniques. In addition, the introduction of high-speed passenger trains in Japan, France, and Great Britain is extending the state of the art of bridge design. An examination of some specific installations illuminates these developments.

3. BART, A CASE STUDY

Prestressed concrete was introduced in the twenties but did not become widely used in bridge structures until the late forties and early fifties. The first large transit project to make extensive use of prestressed concrete was the Bay Area Rapid Transit (BART) system in the San Francisco Bay area of the United States. The BART project assembles many state-of-the-art transit technologies into a single project. Consequently, the lessons learned in its development are useful to modern transit systems.

3.1 Precast Concrete Structural Systems

The BART elevated guideway is a series of simple span beams with directly fixed continuously welded rails placed continuously along the structure and across the structural expansion joints. The project was of sufficient size that the designers concluded a new cost-effective precast concrete structural element could be designed and fabricated specifically for this project. The resulting beams are fully integrated structural elements that were delivered to the job site, ready for erection and installation of final trackwork.

3.2 Continuous Welded Rail Placement

Trackwork consists of continuously welded rails mounted on rail fasteners that are cast in a second pour into a groove on the beam deck. This procedure assures adequate bonding of the rail anchors with the parent beam. A second pour was determined essential on the BART project to assure that the electrical isolation between the rails and the structure is maintained. Running rails are used for the electrical ground return system.

Use of continuously welded rails on simple span beams creates thermal strain discontinuities at the beam expansion joints. The resulting forces must be accommodated into the structure. Accumulation of these forces in longer structures creates severe structural difficulties that will be discussed in the Vancouver Automated Light Rail Transit (ALRT) project.

3.3 Automatic Train Control

Totally automatic operation of transit systems has a number of subtle influences on the design of transit structures, most associated with safety issues such as vehicle restraint and passenger egress in the event of an accident. One early discovery in the BART project was that safe stopping distances must be programmed into the vehicle control system. This required that engineers be able to predict vehicle decelerations accurately enough to support control system assumptions. Using steel wheels with conventional braking systems, the coefficient of friction between the steel rails and the steel wheels under all environmental conditions had to be determined, a more difficult issue than originally anticipated. It is this uncertainty of traction that gave impetus to development of automated rubber tire transit systems.

Figure 4 The Metropolitan Toronto Zoo Guideway used precast concrete double tee beams to achieve a low-cost guideway system. A cast-in-place topping provides continuity and steering curb. (Photo by author)

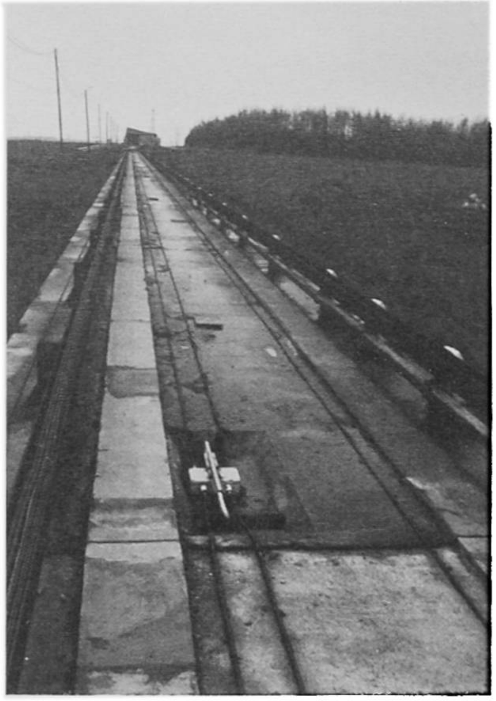

Figure 5 The Matra-Val Transit System in Lille, France, uses a precast concrete running pad for its at-grade and elevated guideway. The running pads can be equipped with heating cables for operation in ice and snow environments. Steel I-beams at the side of the guideway provide a steering surface and serve as power distribution rails. (Photo by author)

Figure 6 Busch Gardens in Tampa, Florida, uses an I-beam guideway to support a suspended vehicle. Low operating speeds allow use of a very small guideway section. (Photo by Dr. Robert Stevens, used with permission)

4.0 RUBBER TIRE TRANSIT

Rubber tire transit systems were proposed for slow-to intermediate-speed transit applications to take advantage of the increased coefficient of friction between the rubber tire and the support surface. Rubber tire systems use concrete as the traction surface. Several systems use steel superstructures with composite concrete top surfaces and, occasionally, steel beams are used with an abrasive material bonded to the steel for traction. For the sake of discussion, the rubber tire systems have been divided into two categories: Conventional support and monorails. Conventionally supported systems use structures similar to a bridge to carry the vehicle. Monorails refer to the type of system where the vehicle straddles a single beam.

Both conventional and monorail systems must address the issue of vehicle steering. It is this steering function and associated switching problems that differentiate the transit guideway from the conventional bridge. In addition to resolving the vehicle steering interaction with the guideway, the designer must also examine associated issues such as wayside power, control systems, switch hardware placement, emergency egress planning, and geometric control for rider comfort.

4.1 Conventional Systems

These systems are so named because the vehicles, except for unique features necessary for system operations, resemble small buses. They are supported from below. Figures 2 through 7 describe several different rubber tire transit systems and illustrate many of the systems' features. The photographs also depict the wide range of structural engineering solutions available for specialized transit systems.

4.2 Monorails

Monorails represent a special subset of rubber tire transit systems. Monorails have been in existence for over a century; however, the straddle-type monorail has been in commercial existence only since the early fifties. One of the earliest commercial applications was at Disneyland in Anaheim, California. Figures 6 and 7 show amusement park monorails that are designed for small vehicles and low-speed operation. The first public transit monorail was constructed in Seattle, Washington, for the 1962 World's Fair. Other mass transit monorails are in use in Japan. The Walt Disney World-EPCOT complex in Orlando, Florida, uses over 10 miles (16 kilometers) of monorail as a principal transportation link between major activity centers and is typical of the type of monorail in mass transit service (Figure 8).

A derivitive of the monorail developed in the late sixties and early seventies used air cushions as its principal means of suspension. The French Aerotrain and the British Hovercraft were the two most advanced prototypes of this type of monorail. Neither project was developed past the prototypical test phase.

The simplicity of a monorail structure differentiates it from other transit structures. The total structure is combined into a single load carrying member. Structurally, the interaction of shear, torsion, and flexural moment creates an exacting engineering condition. Thus, the engineer is obliged to perform a detailed design investigation of all the possible interacting loads which may occur. The lack of redundant load paths and the relatively small size of the structural members make the design one of true structural optimization.

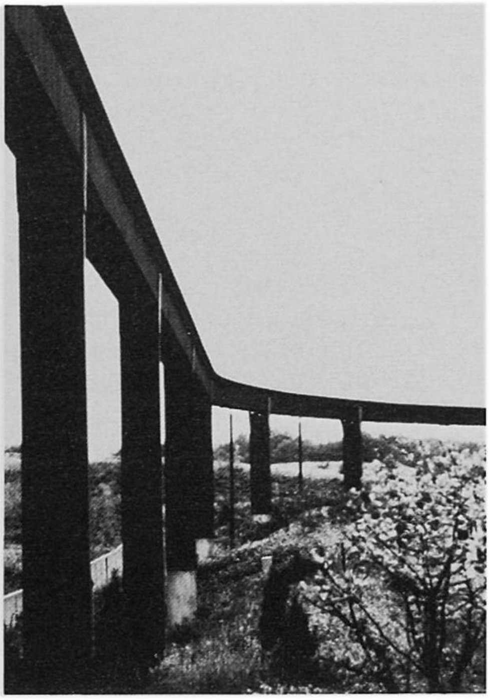

Figure 7 The Minneapolis Zoo uses a small steel box girder to support a lightweight low-velocity train. (Photo by M. LaNier, used with permission)

Figure 8 The Walt Disney World Monorail uses precast concrete beams and a flexible steel plate to connect the beams to the columns. The resulting structure is virtually maintenance free and close to an optimum structural design. (Photo c. Walt Disney Productions, used with permission)

5. EMERGING TECHNOLOGIES

New and exotic technologies are emerging for use in transit applications and are having a pronounced effect on the degree of sophistication of associated structural design. The majority of current development appears to be focused on linear motor technology and magnetic levitation systems.

5.1 Linear Induction Motor Systems

The Urban Transportation Development Corporation (UTDC) of Toronto, Canada, has developed a lightweight intermediate capacity transit vehicle which runs on continuously welded steel rails and is powered by a LIM. The structural development of two- and three-span structures was required to allow the transit system to operate in an urban setting with short radius turning requirements. In turn, these structures have required advancements in state-of-the-art rail structure interaction analysis. Secondary forces resulting from the relative motion between the continuous rails and the continuous structure during changes in ambient temperature become significant. Horizontal curve radii as small as 115 feet (35 meters) generate thermally induced rail forces on the structure which can become the dominant design condition.

The design rationale of the UTDC system includes extensive consideration of urban integration concerns such as noise and visual intrusion, and life-cycle operation costs of the transit system. To this extent, the high capital costs of direct fixation running rails and LIMs are traded off against lower projected operation and maintenance costs. This trade-off resulted in the use of continuously welded running rails and a steerable vehicle bogie. Since acceleration and deceleration are accomplished by the LIM, there is little running rail wear and no dependence upon mechanical friction. This combination produces a 25-year useful life for the running rails.

The first commercial installation of a UTDC system is the Vancouver, Canada, ALRT system. In addition to the LIM propulsion, a second feature of the transit structure is the fabrication of precast guideway beams to their final geometry, complete with inserts for installation of running rail fasteners. This process is projected to substantially reduce the site construction time and to result in superior trackwork dimensional stability.

The German Cabinentaxi is another application of LIM technology for transit application. The Cabinentaxi system uses small 3- to 20-passenger vehicles traveling in a closed loop. A unique feature of this system is the configuration of the vehicle such that half the fleet operates on top of the beam while the other half is suspended below. The resulting structure is extremely efficient and requires little more than 3 feet (1 meter) of width for a dual-lane transit system.

5.2 Magnetic Levitation Systems

The German and Japanese governments are sponsoring research and development of new transit technologies based on magnetic levitation suspension systems. Magnetic levitation (MagLev) technology uses magnetic attraction or repulsion to support, guide, and propel the transit vehicle. MagLev offers the potential for very high-speed operation at low energy consumption levels. Operational speeds of 155 miles per hour (250 kilometers per hour) would make MagLev competitive with air travel for short-haul operations.

To achieve projected energy use and speed potentials, the vehicle must operate with a very small separation between the vehicle and the guideway reaction rails. A paramount factor in the structure design is the installation and

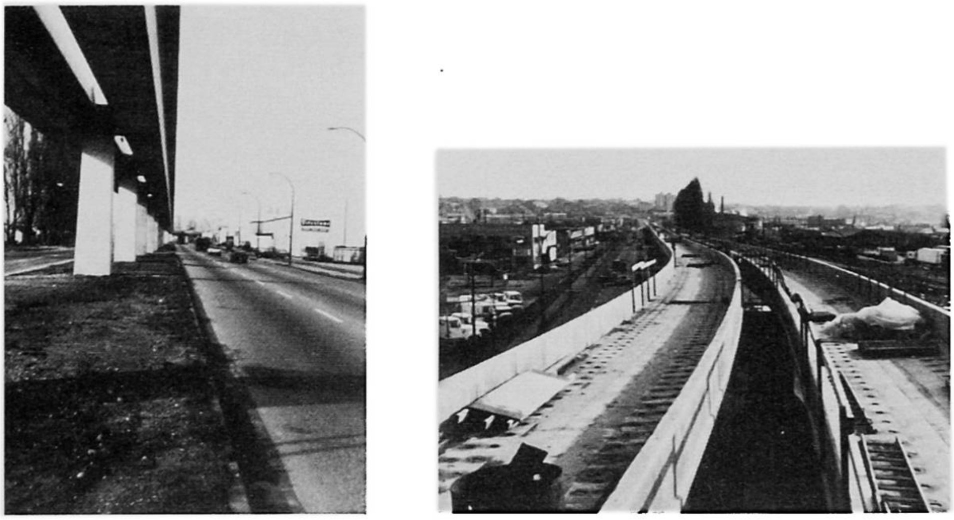

Figure 9 The Vancouver Advanced Light Rapid Transit System uses linear induction motors and continuously welded rails. Left is a photo of the completed structure from below, and right shows the beams during construction. (Photos by ABAM Engineers)

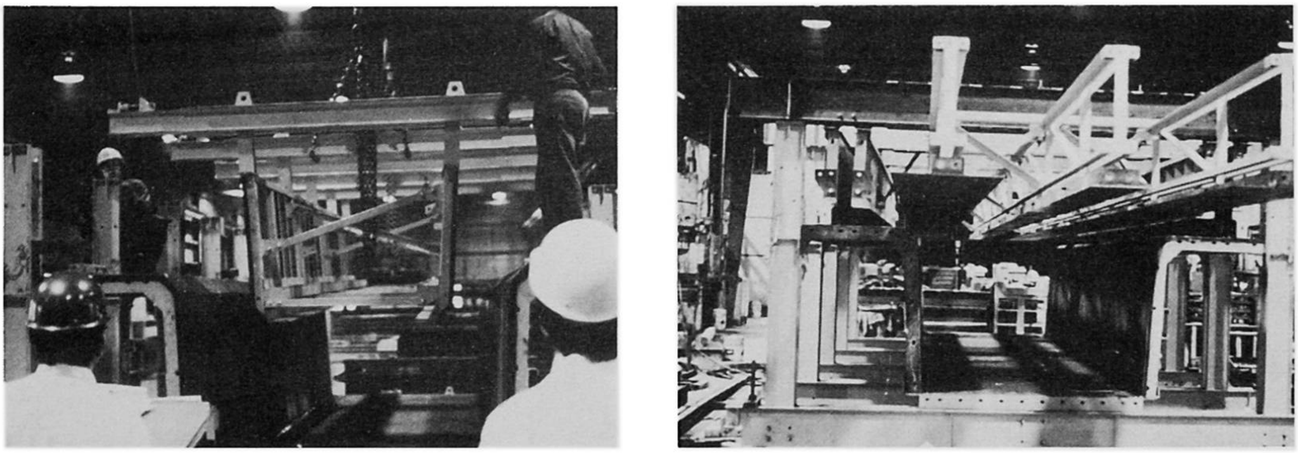

Figure 10 Adjustable forms allow the Vancouver beams to be manufactured to final geometry and tolerance. Left is the form and inner liner, and right is the form and the insert jig mounting frame. (Photos by ABAM Engineers)

maintenance of very tight reaction rail tolerances on the guideway. On prototypical structures now under construction in Germany, beam formwork tolerances of ±0.039 inch in 82 feet (±1 millimeter in 25.6 meters) are being reported for the precast concrete beams.

6.0 CONCLUSIONS

A great need exists to develop new and innovative transit structures to complement existing and emerging transit technologies in order to meet the cost, constructability, and service requirements of transit systems in the world's cities. Transit structures present the structural designer and contractor with unprecedented opportunities for development of creative designs and construction techniques.

7.0 REFERENCES

An extensive bibliography of transit-related articles and data can be found in the following reports.

1. State-of-the-Art Report on Concrete Guideways. Concrete International, American Concrete Institute, Vol. 2, No. 7, July 1980.

2. Steel Structures for Mass Transit. American Iron and Steel Institute, New York, 1975.

Seminar IV

Thermal Performance of Buildings

Comportement thermique des bâtiments

Wärmetechnisches Verhalten von Gebäuden

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Ken-ichi Kimura, born in 1933, graduated from Waseda University in Tokyo in 1957. Took Dr. Eng. degree in 1965. Study abroad to MIT, USA and NRC, Canada. At Waseda Univ. Associate Prof. in 1967 and Professor in 1973. He has been engaged in various consulting works for building facilities.

SUMMARY
The thermal performance of buildings is one of the most important considerations both in the architectural as well as the structural design of buildings. The paper cites several examples of details and modifications which when properly designed can significantly increase the thermal performance of buildings and thereby reduce the heating and cooling energy demands of such structures. Among them are fenestration details, thermal bridges, heat storage, and solar heating and shading.

RESUME
Le comportement thermique des bâtiments est un des aspects les plus essentiels dans le projet architectural et structural de bâtiments. L'article donne plusieurs exemples de détails constructifs et de modifications qui, correctement réalisés, peuvent augmenter de façon significative le comportement thermique de bâtiments; les besoins en énergie de chauffage et de refroidissement de telles structures s'en trouvent ainsi réduits. Le rapport traite également des détails de fenêtres, de ponts thermiques, d'accumulation d'énergie, de chauffage par énergie solaire et d'écrans de protection solaire.

ZUSAMMENFASSUNG
Das wärmetechnische Verhalten von Gebäuden ist eines der wichtigsten Kriterien beim architektonischen sowie strukturellen Entwurf von Gebäuden. Es wird über mehrere Beispiele von konstruktiven Details und Änderungen berichtet, welche, wenn korrekt entworfen und ausgeführt, das wärmetechnische Verhalten von Gebäuden bedeutend verbessern. Damit werden die Energieansprüche für Heizung und Kühlung von Gebäuden reduziert. Es wird auch über Fensterdetails, thermische Brücken, Wärmespeicherung, Sonnenenergie und Sonnenschutz berichtet.

INTRODUCTION

The thermal performance of buildings is one of the most important considerations not only in architectural design of buildings (or system design of building services) but also in structural design. In the recent energy crisis, various measures for energy conservation have been included in architectural design to reduce the heating and cooling loads of buildings. Thermal insulation for windows, walls and roofs, air tightness of windows and proper shape of sun shades are considered to be the principal factors of an energy conserving design. On the other hand there are numerous features of thermal performance of buildings would affect structural design. The fact that most of the participants in IABSE meetings are structural engineers, makes such papers relating to thermal performance of building structures as a whole or building components welcome and called upon. This is because a sound approach to structural design with respect to thermal performance of buildings is important to stimulate structural engineers. Additional emphasis should be placed on the thermal performance of buildings in relation to air conditioning system design. The following are some examples of the themes considered interesting and informative for the Building Physics Session newly installed at IABSE Vancouver Conference.

1. EXPANSION AND CONTRACTION OF BUILDING STRUCTURES

The main frame of tall buildings may not suffer serious damage from thermal expansion and contraction, but it may cause a lot of problems where different materials meet in various parts of the building. Large, long structures usually require thermal expansion joints to accomodate possible expansion and contraction. Measures for absorbing thermal stresses within the structure are generally used, but the effects on minor parts such as bolts have not been clarified in terms of fatigue and other unexpected damage.

2. LOCAL THERMAL STRESS IN FENESTRATION

Breakage of glass panels sometimes occurs due to local thermal stress producing a hazard to people at large. The local thermal stresses are caused by non-uniform thermal pressures on different parts of glass panes. For example, the sunlit part of a glass pane will tend to expand more than the shaded part. This is likely to occur in heat absorbing glass on a very cold day. In order to avoid this, care must be taken in designing the joint details where a glass pane meets the framing to allow for expansion and contraction. Theoretical and experimental studies would enforce the practicability for safety in fenestration design.

3. THERMAL BRIDGES

Structural members often act as thermal bridges. This has not been of too much concern when people were not conscious of energy problems. However, in the age of energy conservation just as building insulation is regarded very important in every part of the exterior surface of a building, it is being recognized more strongly than before that uninsulated portions of structural members can give rise to considerable heat loss. It is obvious that such heat loss would impair the effect of heavy insulation in a wall, for example, despite the fact that the surface area occupied by the structural members is only a small portion of the total surface area of the wall. Uninsulated parts of structural members also allow condensation of water on their inside surfaces giving rise to unexpected damage. In particular, condensation on the surface of exterior walls within the plenum ceiling space is unrecognizable and likely to occur unless proper treatment was made.

4. HEAT STORAGE TANK

The space beneath the basement floor is often used as a heat storage tank or thermal reservoir. In large buildings basement girders are so large that the space between girders can be used to store large amounts of water. In order to shift the peak demand of electricity, cold energy can be stored in the tanks by operating refrigeration machines with off peak electricity at lower rates. The thermal performance of such heat storage tanks on a dynamic basis has been studied quite extensively in accordance with heating and cooling load variations in the room. It is interesting to control the operation of an air conditioning system by predicting the load pattern for the following day.

5. THERMAL MASS EFFECT OF BUILDING STRUCTURES

It is generally known that heavy structures tend to have more stable temperatures in the interior spaces than light weight constructions. Tall buildings are more likely to be constructed with steel frames and columus and light weight floor slabs, thus disploying a rather rapid thermal response. The room air temperature in such a building is apt to rise and fall quite quickly in accordance with heat supply and extraction. The heavy structure in massive buildings, on the other hand, responds slowly to thermal impact. Calculation of room air temperature variation taking account such thermal behaviour has been extensively made in recent years, as owners of buildings have become conscious of the annual energy requirements for heating and cooling. Optimum control strategies using computers have been investigated and realized in some of the advanced examples. Studies are required to explain the method of selection of a desirable thermal performance in terms of heat capacity of building structures for tall buildings. A structure with low mass but high thermal inertia might possibly be desirable.

6. SOLAR HEAT GAIN AND HEAT LOSS FROM WINDOWS

Windows must be designed so as to promote solar heat gain in winter and discourage it in summer as well as for daylight and ventilation. Windows can be considered as solar heat collectors in themselves if thermal storage devices are provided in the interior spaces. In colder countries, double or triple glazing is justified to reduce heat loss in winter. In warmer countries where the cooling season dominates, however, insulated windows often interfere with heat dissipation from inside to outside during the night, because a considerable amount of heat generated from lights, electrical appliances and occupants during the daytime is accumulated within the building structure unless other means of night purge ventilation are provided. Thermal balance between these two situations has not been clearly determined, as this must be quantitatively evaluated on an annual basis.

7. SOLAR SHADING DEVICES ON FENESTRATION

Solar shading devices provided on the outside of fenestration is often referred to as "brise-soleil" especially when they are designed to be an integral part of the main structure of a building. The thermal performance of intercepting direct solar radiation to reduce the cooling load has been analyzed and indicates a different optimum configuration for different localities. The sunlit members of shading devices might cause excessive thermal stresses due to repetition of temperature changes between day and night particularly in hot countries. The supporting structures are also very important as they are often installed along streets. Special care must be taken when the buildings are to be built in earthquake prone countries, lest they sould fall on the street.

8. SUMMARY

The topics discussed above are some of the examples of details effecting the thermal performance of buildings which might attract the interests of structural engineers as well as scientists of building physicists. There must also be many other interesting topics in this area to stimulate participation. Interaction between the different specialists are particularly important for enhancement of the quality of buildings and the IABSE Vancouver Conference is expected to provide a forum to open fruitful discussion for the future of building technology.

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Seminar V

Fortschritte in Entwurf und Berechnung von Stahltragwerken

Developments in the Design of Steel Structures

Développements dans le projet et le calcul de constructions métialliques

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ZUSAMMENFASSUNG
Anhand von Beispielen wird auf Entwicklungen und Tendenzen in Entwurf und Berechnung von Stahltragwerken eingegangen, wie sie sich auch in den neuesten ISO- und Eurocode-Entwürfen für die Bemessung und Berechnung von Stahltragwerken niederschlagen.

SUMMARY
The article presents, with examples, the developments and trends in the design of steel structures. It considers also the latest drafts of the ISO and Eurocode recommendations for the design of steel structures.

RESUME
L'article considère, à l'aide d'exemples, les développements et les tendances dans le projet et le calcul de constructions métalliques. Cette évolution se reflète également dans les projets les plus récents des normes ISO et Eurocode pour le projet et le calcul de constructions métialliques.

1. ALLGEMEINES

Die Sicherheitsnachweise für Stahltragwerke sind zur Zeit weltweit in Umstellung begriffen; von dem bislang verwendeten Verfahren der zulässigen Spannungen wird auf das sogenannte semiprobabilistische Verfahren mit getrennten Nachweisen für die Grenzzustände der Gebrauchsfähigkeit und der Tragfähigkeit umgestellt, und dabei werden die Grenzzustände und die dafür anzuwendenden Einwirkungen und Einwirkungskombinationen mit den zugehörigen Sicherheitskoeffizienten neu definiert.

Dies geschieht zum Zwecke der Harminisierung des Nachweisformates über alle Bauweisen und Baustoffe hinweg und bietet für neue Bauweisen und Baustoffe den Vorteil einer vergleichbaren sicherheitsmäßigen Einordnung; es wird aber durch den Bezug auf eine definierte Zielzuverlässigkeit für die Tragfähigkeit und für die Gebrauchsfähigkeit auch beabsichtigt, die Bemessungssicherheit und die wirtschaftliche Ausnutzung des Materials im Sinne einer Konstruktionsoptimierung zu vergleichmäßigen. Schließlich wird durch ausdrückliches Offenlegen aller Gefährdungen und durch die Nachweise ihrer Begegnung eine sichere Entwurfsarbeit und eine bessere Baudurchführung angestrebt.

Die Umstellung auf das neue Nachweisformat ist durch die Arbeit einer Reihe internationaler technisch-wissenschaftlicher Verbände vorbereitet, die wichtige Berichte und Empfehlungen herausgegeben haben. Erste Entwürfe für internationale Standardempfehlungen für Stahlkonstruktionen erschienen bereits bei ISO und im Rahmen der Europäischen Gemeinschaften als Eurocodes. Diese internationalen Entwürfe sind naturgemäß umso allgemeiner gehalten, je mehr Mitgliedsstaaten mitwirkten; sie gehen häufig nationalen Normenarbeiten voraus und werden in Zukunft eine größere Vereinheitlichung der Vorgehensweisen bei Entwurf und Bemessung von Stahlkonstruktionen bewirken.

2. BESONDERE BEDINGUNGEN FÜR ENTWURFS- UND BEMESSUNGSREGELN IM STAHLBAU

Zunächst gilt als Feststellung, daß bei den meisten Bauaufgaben im Brücken- und Hochbau die Planung und die Realisierung als Stahlbau nicht mehr den Regel-, sondern mehr den Sonderfall darstellt, d. h. Consultings und Behörden haben relativ selten mit Stahlkonstruktionen zu tun. Es muß deshalb im Interesse des Stahlbaus liegen, die Entwurfs- und Berechnungsmethoden möglichst so an geläufige Methoden anzupassen und einfach darzustellen, daß Stahlbauten leicht von Ingenieuren entworfen und beurteilt werden können, die gewöhnlich mit Massivbauten zu tun haben. Ein hochspezialisierter Stahlbauingenieur ist in der Regel in Büros nicht vorhanden, und die Antwort von Consultings und Behörden auf ungewohnte, kompliziert erscheinende Entwürfs- und Bemessungsregeln würde sein, daß Stahl nur da eingesetzt wird, wo es nicht anders geht; die Bearbeitung von Alternativentwürfen in Stahl, die zu einer breiteren Anwendung führen sollen, würden erschwert.

Ein weiteres Argument für einfache und durchschaubare Regeln liegt in der Struktur der Stahlbauunternehmen. Dazu gehören häufig Klein- und Mittelbetriebe mit lokalem Marktfeld, die aus Wirtschaftlichkeitsgründen mit kleinem Entwurfsbüro arbeiten, das nicht mit übermäßigen Berechnungen blockiert werden kann. Natürlich müssen einfache Regeln sicher und wirtschaftlich sein und deshalb auf der Basis realitätsnaher und daher oft komplizierter Modelle entwickelt werden.

Aus diesen Bedingungen ergibt sich notgedrungen ein zweispuriges Vorgehen bei der Entwicklung von Entwurfs- und Bemessungsregeln:

Auf der einen Seite geht es darum, für die Anwendung von Computern in der Entwurfsarbeit kompliziertere Modelle beliebiger Annäherung benutzen zu können, wie zum Beispiel Traglastmodelle Theorie 2.Ordnung mit Berücksichtigung elastisch-plastischen Werkstoffverhaltens, wofür Vereinbarungen über die Rechen-

annahmen auf der Lastseite und auf der Widerstandsseite erforderlich sind, wie zum Beispiel
- Spannungs- und Dehnungslinie für der Stahl
- geometrische Imperfektionen inform von Schiefstellungen, Stabkrümmungen oder Vorbeulen
- Ansatz von Eigenspannungsfeldern
- Last-Verformungsverhalten von Anschlüssen.

Solche Modelle finden in der wissenschaftlichen Arbeit, zum Beispiel zum Studium von Phänomenen und zur Ableitung und Begründung einfacher Verfahren, aber auch zunehmend in der praktischen Anwendung, zum Beispiel bei Sondertragwerken mit besonderen Sicherheitsproblemen, und bei entsprechend schnell-laufenden guten Programmen auch in der breiteren Entwurfsarbeit Verwendung.

Auf der anderen Seite sind einfache, möglichst lineare oder quasilineare Modelle und Beziehungen für die normale Entwurfsarbeit notwendig; hierbei müssen die Anwendungsgrenzen definiert sein, und im Falle der Überschreitung müssen einfache Abschätzverfahren zur Berücksichtigung von Nichtlinearitäten anwendbar sein. Die Anwendung dieser vereinfachten Verfahren erstreckt sich vom Tischcomputer bis zur Handhabung mit kleinem Taschenrechner, und es sollte möglich sein, sozusagen aus der Aktentasche heraus einen Entwurf durchzuführen und zu kontrollieren.

3. ZUM TRAGFÄHIGKEITSNACHWEIS

3.1 Zugstab

Grenzzustände der Tragfähigkeit zu definieren, kann selbst bei einem so einfachen Bauelement wie dem Zugstab zu unterschiedlichen Ergebnissen führen.

Konventionell wird nicht die Bruchgrenze des Zugstabes als Grenzzustand definiert, sondern ein Zustand, der bei den üblichen Baustählen weit unterhalb der Bruchgrenze liegt (ca. 70 %), nämlich am Beginn erkennbarer plastischer Verformungen hervorgerufen durch Fließen. Ist der Zugstab gestört, zum Beispiel durch Löcher an den Anschlüssen, so sind die Bemessungs- und Detailierungsregeln so, daß immer das Fließen des Stabes gewährleistet ist, bevor an Störstellen Bruch auftritt. Diese als Duktilität bezeichnete besondere Eigenschaft von Stahlkonstruktionen kann als Bemessungsphilosophie auch auf Schraubverbindungen und Schweißverbindungen übertragen werden. Wenn es die Sicherheitsbedingungen verlangen oder besondere Duktilitätsanforderungen bestehen, zum Beispiel für die Energiedissipation für Erdbeben, können die Schrauben nicht nur für ausreichende Tragfähigkeit gegenüber den Anschlußkräften bemessen werden, die sich aus den rechnerischen Lasten ergeben, sondern derart, daß Fließen des Zugstabes immer gewährleistet ist, bevor die Schrauben brechen. Dies bedeutet nicht nur Einhaltung einer Mindestfließgrenze, sondern die Kenntnis der Höchstfließgrenze des Zugstabes.

3.2 Druckbeanspruchte Bauteile

Die Festlegung der Tragfähigkeit von Druckstäben oder Biegeträgern mit druckbeanspruchten Teilen ist nicht am Spannungsdehnungsdiagramm des Kleinteilversuches, sondern an Bauteilversuchen im Maßstab 1 : 1 orientiert.

Die Versuche lieferten für die Grenzzustände zwei wichtige Größen: einmal die Werte der größten aufnehmbaren Last, der sogenannten Traglast, die durch verschiedene Stabilitätsphänomene begrenzt sein kann, wie zum Beispiel lokales Beulen, Biegeknicken oder Biegedrillknicken; zum anderen den Verformungsbereich, in dem das Bauteil die Traglast auf gleichem Niveau halten kann.

Aus der statistischen Analyse der gemessenen Größen der Traglast sind die charakteristischen Werte der Festigkeit ableitbar, zum Beispiel die Europäischen Knickspannungskurven. Indem man zusätzliche Sicherheitselemente inform von Eigenspannungsverteilungen und/oder geometrischer Imperfektionen am statischen Modell des Stabes oder des Stabsystems einführte, konnten die gemessenen Festigkeitswerte auch rechnerisch als Grenzschnittgrößen des verformten Systems erklärt werden. Daraus hat sich die Möglichkeit entwickelt, Stabilitätsprobleme grundsätzlich nicht mehr durch Vergleich mit den theoretischen Verzweigungslasten, sondern realistischer durch Nachweis des Grenzzustandes für imperfekte Strukturen nach der Theorie 2. Ordnung (nicht linearem geometrischen Modell) zu lösen.

Die Einteilung der Stauchungsfähigkeit von Querschnitten auf dem Traglastniveau in verschiedene Klassen ist eine Hilfe für die Anwendung verschiedener Materialmodelle bei der Berechnung der Traglast; nämlich eines linearen Materialmodells, bei dem die Verformungen als elastisch unterstellt und eine Stauchung bis zur Beulgrenze oder bis in den Fließbereich zugelassen wird oder eines nicht linearen Materialmodells, bei dem aufgrund größerer Verformungsfähigkeit planmäßig Schnittgrößenumlagerungen aus Fließzonenausbreitungen, auch vereinfacht dargestellt als Wirkung von "Fließ gelenken" berücksichtigt werden können.

Um der Praxis zu ersparen, jedes Problem mit nicht linearem geometrischem Modell (Theorie 2. Ordnung) berechnen zu müssen und um die Vorteile der Querschnitts- und Systemtragfähigkeit infolge des Fließens auch ohne größeren Rechenaufwand in Anspruch nehmen zu können, sind Vereinfachungen vereinbart worden. Danach dürfen Stabwerke grundsätzlich nach Theorie 1. Ordnung berechnet werden, wenn nicht bestimmte Kriterien für die Horizontalsteifigkeit von Strukturen und Stabsteifigkeit von Stäben überschritten werden. Bei Überschreitung dieser Kriterien werden zusätzlich Näherungslösungen auf der Basis der Ergebnisse der Theorie 1. Ordnung angegeben.

Für Einzelstäbe sind Gebrauchsformeln für den Nachweis des Biegeknickens und Biegedrillknickens unter beliebiger Randbelastung abgeleitet worden, mit denen die aus Strukturen herausgeschnittenen Einzelstäbe nachgewiesen werden können und die sich auch für den Nachweis ganzer Strukturen eignen, wenn deren Verhalten auf das Verhalten von Einzelstäben zurückgeführt werden kann, zum Beispiel durch Knicklängen.

Besondere Bemessungsregeln sind zur Zeit noch häufig für Bauteile aus dünnwandigen kaltverformten Profilen und Blechen erforderlich, deren wirtschaftliche Bedeutung zunehmend wächst. Die Bemessung könnte in Zukunft mit Vorteil an die Bemessungsregeln für konventionelle Stabelemente aus Walzprofilen oder geschweißten Profilen angepaßt werden, und nur die zusätzlichen Versagensmodi für dünnwandige Elemente müßten getrennt berücksichtigt werden. Eine Vereinheitlichung der Nachweise für dünnwandige und nicht dünnwandige Bauteile würde auch die Beulnachweise für ausgesteifte und nicht ausgesteifte Blechfelder betreffen. Auf diesem Sektor gibt es eine Reihe verschiedenster Definitionen von Grenzzuständen und verschiedener Bemessungsmodelle.

3.3 VERBINDUNGSMITTEL UND VERBINDUNGEN

Eine weitgehende Anpassung der Regeln ist auch für die Bemessung von Verbindungsmitteln zu erwarten.
Das Tragverhalten von Schraubenverbindungen ist für Baustähle bis Festigkeiten ähnlich Fe 510 unter statischer Last genügend geklärt und die Regeln sind soweit entwickelt, daß Vereinfachungen zum Beispiel für Schraubenbestellungen (Konzentration auf weniger verschiedene Materialgüten, Eingrenzung unterschiedlicher Klemmlängen durch Zulassung von Gewinde im Loch etc.) möglich sind. Besonders wichtig sind die erreichten Vereinfachungen der konstruktiven Detailierung von Schraub- und von Schweißverbindungen als "steifenlose Verbindungen" für statische Lasten und der entsprechenden Nachweise zum

Beispiel für Kopfplatten, Auflager- und Anschlußpunkte.

Für oft wiederholte Belastungen sind geschraubte und geschweißte Anschlußstellen als Störstellen des Kraftflusses hinsichtlich ihrer Betriebsfestigkeit zu untersuchen, wofür ein einfaches Festigkeitsmodell inform der Europäischen Ermüdungslinien für verschiedene Kerbkategorien entwickelt wurde, das eine relativ einfache Betriebsfestigkeitsüberprüfung erlaubt. Hier werden in Zukunft Entwicklungen vor allem bei den Vereinfachungen der Belastungsbeschreibung zu erwarten sein.

Eine trendtypische Entwicklung stellen im Zusammenhang mit Verbindungen knotenblechfreie Fachwerkkonstruktionen mit Hohlprofilen dar, bei denen die wirtschaftliche steifenlose Ausbildung der geschweißten Knoten besondere Nachweise für statische und wiederholte Belastung erfordert, die für die Anwendung stark vereinfacht werden konnten. Typisch für wirtschaftliche Anschlußlösungen ist hierbei die Entfeinerung der Verbindungen durch Inkaufnahme von Exzentrizitäten der geometrischen Achsen (weg vom idealen Fachwerk) und ungleichmäßiger Steifigkeits- und Kraftverteilungen längs der Anschlußfugen.

3.4 KONTROLLEN

Das den Funktionszielen und Festigkeitsannahmen zugrunde gelegte Qualitätsniveau für die Ausführung von Strukturen, Bauelementen und von Anschlüssen wird durch Toleranzen für Imperfektionen und Defekte festgelegt, deren Einhaltung durch Kontrollen verschiedener Intensität nachgewiesen werden muß (Qualitätssicherung). Hier sind noch bessere Zusammenhänge zwischen Defekten und zugehöriger Festigkeit in Zukunft erwünscht.

4. ZUM GEBRAUCHSFÄHIGKEITSNACHWEIS

Während bei der Anwendung des früheren Bemessungsverfahrens mit zulässigen Spannungen häufig neben der Tragsicherheit auch eine ausreichende Gebrauchssicherheit sichergestellt war, sind infolge der Ausnutzung der Fließreserven in Querschnitten und Strukturen bei dem neuen Tragsicherheitsnachweis die Bedingungen für ordnungsgemäße Funktion nicht automatisch erfüllt.

Demzufolge ist die Erfüllung dieser Bedingungen auf dem Niveau der Gebrauchslasten gesondert nachzuweisen. Das bedeutet einen gewissen Mehraufwand. Der Gebrauchsfähigkeitsnachweis ist aber gerade für Stahlkonstruktionen, denen im Wettbewerb häufig nachgesagt wird, sie seien zu "weich", neigen zu Schwingungen etc., besonders wichtig und ist häufig für die Bemessung maßgebend. Die Gebrauchsbedingungen betreffen Funktionsaspekte, zum Beispiel Rissefreiheit von Einbauten in Hochbauten oder von Fahrbahnbelägen auf Brücken, oder Komfortbedingungen zum Beispiel bei Schwingungen von Gebäuden oder Gebäudeteilen, oder Freihalten von Rissen hervorgerufen durch Ermüdung oder auch optische Gesichtspunkte wie störende Verformungen, Atmen von Blechen etc.

Da Schäden an Bauwerken im wesentlichen Schäden sind, die unter Gebrauchsbedingungen auftreten, ist der Nachweis ausreichender Gebrauchstüchtigkeit und Dauerhaftigkeit und damit geringer Unterhaltungskosten für den Bauherrn von besonderer Bedeutung. Er wird ähnlich wie im Massivbau auch im Stahlbau in Zukunft sicher zunehmend ins Blickfeld geraten, möglicherweise mit dem Ergebnis, daß nicht mehr immer der Tragsicherheitsnachweis primär geführt wird.

Die Konsequenzen für die Vereinfachung des Nachweiskonzeptes wären, bei der Definition von Gebrauchslasten und der Sicherheitsbeiwerte, mit denen aus den Gebrauchslasten die Grenzlasten für den Tragfähigkeitsnachweis gebildet werden, so vorzugehen, daß nach Möglichkeit die Durchrechnung von Systemen auf einem Niveau, zum Beispiel dem Gebrauchslastniveau genügt, um damit die Konsequenzen für das zweite Niveau, z. B. das Grenzlastniveau, schnell übersehen zu können.

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Seminar VI

Wind Effects on Structures

Effets du vent sur les structures

Windeinwirkungen auf Tragwerke

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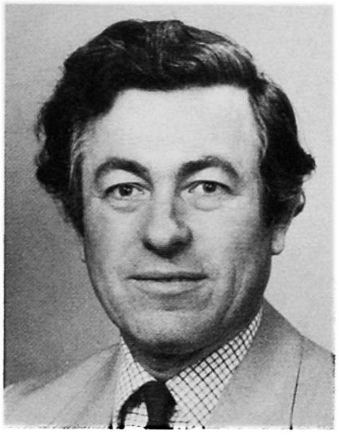

William Melbourne, born in 1934, graduated as an Aeronautical Engineer in 1955 from the University of Sydney and then completed a Ph.D. at Imperial College, London University in 1960. After working for a decade in the aeronautical field he turned to wind engineering research and is currently Chairman of the Department of Mechanical Engineering at Monash University.

SUMMARY
Two concepts, inertial load distributions and use of design wind speeds with direction, both of which are in use by advanced structural designers, are discussed with the objective of illustrating how these concepts can be simply applied and hence incorporated into everyday structural design. The theme is to make progress towards introducing real response and real loading from wind action into the design of structures. The remainder of the paper discusses a number of effects of freestream turbulence on wind loading of structures with the objective of drawing papers dealing with these more complex problems.

RESUME
L'auteur présente deux possibilités pour l'introduction plus réaliste des sollicitations dues au vent: considération des propriétés aéroélastiques de la construction; considération des vitesses effectives en fonction de la direction du vent. Ces deux méthodes sont présentées, l'objectif étant d'illustrer comment elles peuvent être appliquées simplement et par là incorporé dans le travail quotidien de l'ingénieur. Le but est d'introduire la réaction réelle et la charge réelle de l'action du vent dans le projet des structures. L'article discute également un certain nombre d'effets de turbulences et de charges de vent sur les structures, afin de susciter des contributions traitant de ces problèmes plus complexes.

ZUSAMMENFASSUNG
Der Autor zeigt zwei Möglichkeiten zur realistischeren Erfassung der Windbeanspruchung auf: Berücksichtigung der aeroelastischen Eigenschaften des Tragwerkes bei der Einführung der Windlasten; Betrachtung der effektiven Windgeschwindigkeiten in Abhängigkeit der Anströmrichtung. Dabei werden einfache Wege aufgezeigt, um deren Eingang in die tägliche Entwurfsarbeit zu ermöglichen. Ziel ist das effektive Verhalten des Tragwerkes unter den effektiven Windeinwirkungen besser im Entwurf und in der Bemessung zu berücksichtigen. Zum Schluss wird noch eine Anzahl von Einwirkungen infolge Windturbulenzen aufgelistet mit dem Zweck zu diesem komplexeren Problemkreis Beiträge zu erhalten.

1. INTRODUCTION

The last two decades has seen an explosion in knowledge concerning the determination of the response of structures to wind action. By combining work in meteorology, fluid mechanics and structural dynamics, and with the help of probabalistic and spectral mathematics, a description of the real response process has emerged. In spite of this knowledge a great majority of structures, including many major in size and cost, are still designed using wind loads obtained from quasi-steady based wind loading codes. Reliance on such totally artificial load concepts ignores in one direction the economies of scale in reducing global loads and in the other direction the risk of ignoring significant dynamic amplification of response for some wind sensitive structures. This has occurred in a world of structural design where even the smallest structural offices think nothing of using enormously powerful structural packages to analyse member loads. In relative terms it would be a minute effort to add to these programs the ability to output, or use in the calculations, an inertial load distribution related to a given base overturning moment which can now be obtained either from model tests or in many cases analytical techniques for determining response to wind action.

Whilst those of us primarily concerned with wind engineering research have made significant advances in recent years, the impact on the structural design community appears not to have been so significant. It seems that there is still a gap to be bridged to bring the new knowledge and analytical techniques into the world of the practising structural engineer. I will take this opportunity to discuss aspects of new knowledge which could significantly affect the design of structures, in the hope that others at this conference will pick up the theme and make progress towards introducing real response, and real loads, from wind effects on structures into everyday structural design.

2. LOAD DISTRIBUTIONS

In respect of wind loading on towers, chimney stacks and buildings, data from wind tunnel aeroelastic model studies and the more advanced analytical techniques generally finish by determining a peak (design) displacement or base overturning moment. This peak response is made up of a mean and fluctuating component of which the latter makes up 70% or more of the total, and the load distribution which actually stresses components of the structure is consequently primarily a "dynamic" inertial load distribution. That is the load distribution is made up of the sum of the mass elements times the maximum acceleration acting on those elements in a given cycle. When modal analysis programs are run to determine the mode shapes and frequencies it would be a simple matter to include an output of the inertial load distribution for each mode. This would hopefully stop the practice of using load distributions based on quasi-steady wind load distributions which are so commonly used. The very large difference between a typical quasi-steady wind load distribution and the real inertial load distribution can be illustrated by considering a tall cantilevered structure with constant mass per unit length and which for simplicity is taken as oscillating in a cross-wind direction such that the mean response is zero and the peak base overturning moment is all due to the fluctuating component. A typical quasi-steady wind load distribution would be one varying with velocity squared and in which velocity might vary with height, $z^{0.2}$ which gives a load distribution.

$$F(z) = \text{Const } z^{0.4} .$$

The inertial load distribution is a function of the mass multiplied by acceleration and for a constant mass per unit length and cantilever mode shape ($y \propto z^{1.5}$) gives a load distribution

$$F(z) = \text{Const } z^{1.5} .$$

A graphic comparison of these two load distributions, to give the same base overturning moment, is given in Figure 1. It is interesting to note that the mid-height bending moment for the incorrect quasi-steady wind load distribution is just over half the base overturning moment and for the correct inertial load distribution is about two thirds of the base overturning moment. For some cases of towers in chemical plant with heavy elevated vessels this difference can be a factor of two.

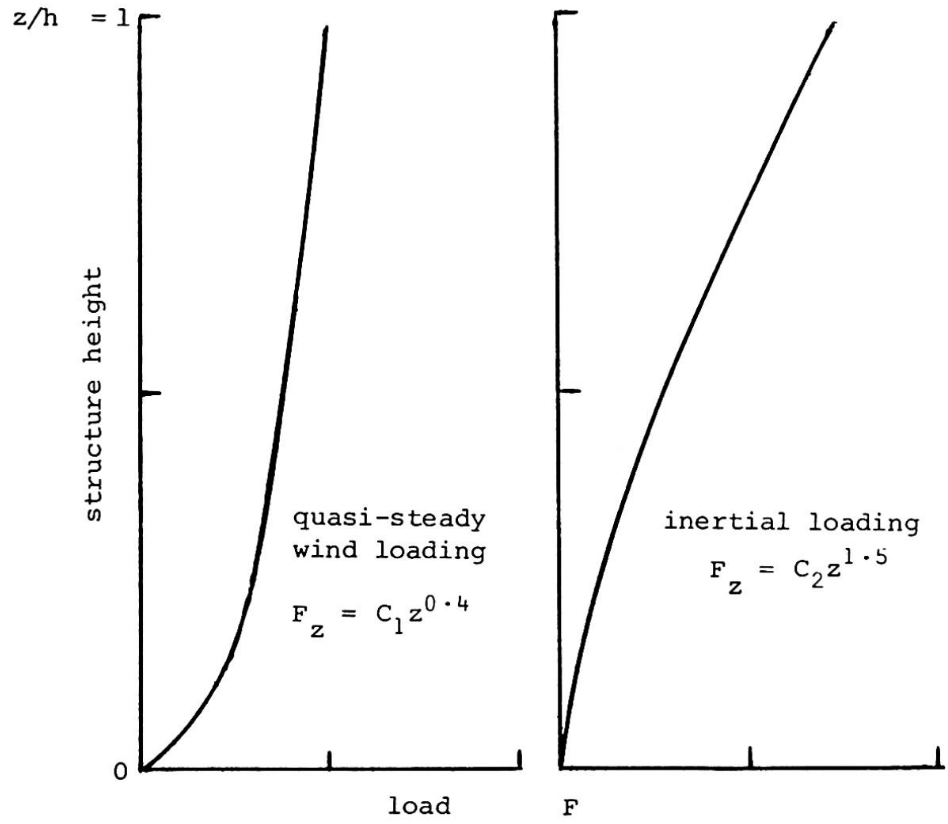

Fig. 1 Comparison of quasi-steady wind load distribution and inertial load distribution for a uniform cantilever tower to give the same base overturning moment

3. WIND DIRECTIONALITY EFFECTS

Wind directionality, or rather the probability distribution of wind speeds with direction has been used in the design of major structures for many years following the pioneering work of Davenport [1]. However we have been slow to apply the same considerations to low rise structures, probably because the apparent sophistication appears not be be worthwhile. A closer look reveals that in many situations there is a potential for design optimisation, and considerable savings from using different values of design wind speed for different directions. For example, a structure may clearly have suburban roughness approaches for all strong wind directions, but have open country, or be exposed on an escarpment for other wind directions. If wind direction cannot be taken into account the design wind speed for the region for all directions has to be used in combination with the worst loading case, and the end result can be a design load up to twice as high as could be rationally required for a given risk. Similar over-design can occur when having to account for a dominant opening when it is facing directions from which the probability of a given design wind speed is much lower.

The problem from the structural engineer's point of view seems to be that he is reluctant to engage in a full probabalistic design approach. However, in regions where there are significant differences in wind speed probability occurrence for broad bands of wind direction, it is possible to devise conservative design wind speeds with direction which can be used in a deterministic design approach without having to undertake a full integration of directional wind speed data with each load case. If the full integration is not undertaken it is essential to devise conservative design wind speeds with direction to account for (a) the additive effects of lower probabilites of other than the design direction (band) under consideration, and (b) the possible defficiencies in determining the directional wind speed probability distribution. In this latter respect reference is intended to the problems of short period directional data available (less than 20 years for example) and in areas where thunderstorms dominate and the only data available are the maximum daily wind speed and direction which can obscure the possibility of a similar but slightly lower wind speed occurring in a neighbouring directional band.

In Australia a pilot attempt has been made to provide simply usable directional wind speed data, and which is currently being tested against a number of design situations prior to making it available for general use. Examples of this will be given here in relation to a low rise factory building in the hope that other authors will be encouraged to provide comment and other examples of the potential savings and problems (pitfalls) which can accrue from using simplified directional design wind speed data.

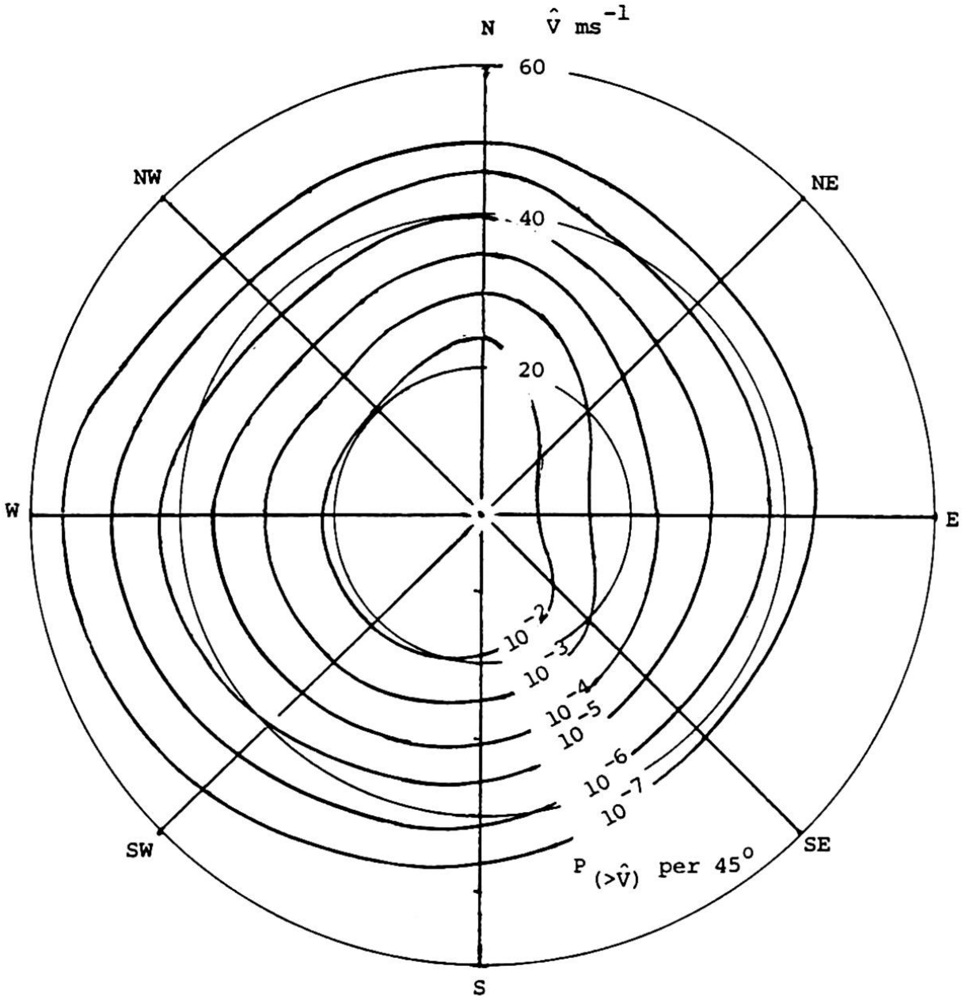

Fig. 2 Probability distribution of 3-second mean maximum wind speeds based on daily maximum data referenced to a height of 10 m in open country terrain (z_o=0.020 m) for the City of Melbourne

In Figure 2 the probability distribution for the 3-second mean maximum wind speeds (gust wind speeds) in the City of Melbourne are given. These data were derived from a composite of 114 years of daily maximum gust wind speeds with direction, from five anemometers spread some 50 km apart (largest single record 32 years). These data have been corrected for approach fetch and anemometer position error for 16 wind directions and given with reference to a 10 m height in open country terrain (roughness length $z_o = 0.020$ m), which is typical of the airfield sites in which most of the anemometers were located. The probability for a 50 year return period is $P(>v) = 1/50 \cdot 365 = 55 \times 10^{-6}$. A full integration for all wind directions gives the 50 year return period wind speed to be 40 ms^{-1}. On evaluation of a number of factors the author has proposed (at least initially) that conservative design directional wind speed data, for use in a simple deterministic design application, be derived for eight wind directions along a contour with half the 50 year return period wind speed probability (in Australia the code requires 50 year return period wind speed to be used in the current factored load design approach). Such an approach results in the design wind speeds with direction given in Table 1. Examples of the application of these design wind speeds, using all directions design wind speed compared with taking direction into account, will be given using pressure coefficients and wind speed profiles for two surface roughness conditions as specified by the Australian Standards Association AS 1170 Part Two, Wind Loads 1981.

Direction	NE	E	SE	S	SW	W	NW	N	All Directions
Wind Velocity	29	27	29	34	38	40	36	38	40

Table 1 REGIONAL BASIC DESIGN WIND VELOCITIES FOR 45° DIRECTIONAL SECTORS FOR THE CITY OF MELBOURNE (3 second mean maximum gust wind speed in ms^{-1} at 10 m height in open terrain, $z_o = 0.020$ m, relating to a 50 year return period)

Consider a low pitch roof on a building, 5 m in height and with a dominant opening in the east wall. From AS1170, th external pressure coefficient, $C_p = -0.9$, and the internal pressure coefficient, $C_p = 0.8$, both acting upwards; and in open country terrain the design gust velocity at 5 m is 0.93 times that at 10 m, and in suburban terrain it is 0.65 that at 10 m in open country terrain (i.e. as referenced in Table 1).

(i) For open country approaches in all directions

(a) For All Direction Design Wind Speed

Design Wind Speed = $40 \cdot 0.93 = 37.2$ ms^{-1}

Design Roof Pressure = $1.7 \cdot 0.6 \cdot 37.2^2 = 1412$ Nm^{-2}

(b) For Directional Design Wind Speed

Design Wind Speed, Max from N, S, and W = $40 \cdot 0.93 = 37.2$ ms^{-1}

Design Wind Speed, Max from NE, E and SE = $29 \cdot 0.93 = 27.0$ ms^{-1}

Design Roof Pressure = $0.9 \cdot 0.6 \cdot 37.2^2 = 747$ Nm^{-2}
or = $1.7 \cdot 0.6 \cdot 27.0^2 = 744$ Nm^{-2}

The former being the design case a reduction of 47% is achieved by taking design wind speed with direction into account.

(ii) For suburban terrain approaches from S, W and N and open country terrain from the E

(a) For All Direction Design Wind Speed

Design Wind Speed $= 40 \cdot 0.93 = 37.2\ ms^{-1}$

Design Roof Pressure $= 1.7 \cdot 0.6 \cdot 37.2^2 = 1412\ Nm^{-2}$

(b) For Directional Design Wind Speeds

Design Wind Speed, Max from N, S and W $= 40 \cdot 0.65 = 26.0\ ms^{-1}$

Design Wind Speed, Max from NE, E and SE $- 29 \cdot 0.93 = 27.0\ ms^{-1}$

Design Roof Pressure $= 0.9 \cdot 0.6 \cdot 26.0^2 = 365\ Nm^{-2}$

or $= 1.7 \cdot 0.6 \cdot 27.0^2 = 743\ Nm^{-2}$

The latter being the design case, a reduction of 47% is achieved by taking design wind speed with direction into account.

If the opening was centrally located in the east wall it would have been reasonable to use the Basic Design Wind Speed for the E Wind Direction of 27 ms^{-1} instead of the 29 ms^{-1} for the NE and SE directions, and this would have resulted in a reduction of 54% by taking wind direction into account.

Obviously there are many examples of the savings in structure which can be achieved by taking wind direction into account. Whilst in the past it has only seemed worthwhile going to the extra trouble for large expensive structures it is hoped that these examples show also how easy it is to achieve significant reductions for a simple, relatively inexpensive structure, but one for which savings are often significant.

4. WIND TURBULENCE EFFECTS

One of the author's main research interests has been in the fundamental aspects of the effect of freestream turbulence on bluff body aerodynamics, and in particular the application of these effects to the design of structures to withstand wind action. To draw on papers which will address these problems it is proposed to highlight a number of recent examples where turbulence effects were very significant in determining design wind loads.

4.1 Cladding loads

There are a number of examples of cladding failure where turbulence plays a major part; the most graphic in recent times have probably been the glass failure on the Boston Hancock Building and the mass failure of roofs in Darwin during the passage of Cyclone Tracy. In both cases there were structural inadequacies which contributed to the scale of the disasters, but in which there were major contributions from the very large enhancement of the high negative pressures under the re-attaching shear layers occasioned by the presence of relatively high turbulence in the incident air stream. It is noted that in low turbulence (smooth) air flows very high intermittent negative pressures near a separating leading edge do not occur at all; it is the presence of the free-stream turbulence which causes the shear layer to re-attach so rapidly on the streamwise surfaces with attendant high negative pressures.

From model tests and limited full scale experience, the highest negative pressures in this respect seem to occur where there is an edge discontinuity (an intersection with a lower stage building or just a change in edge shape) in a region of high freestream turbulence, which, of course, occurs at lower heights near the tops of surrounding buildings or roughness elements. Time and again the highest peak pressures established in model studies come from the lower part of the building, not the higher part, where the mean pressures are always highest. This would not occur if the turbulence characteristics, in particular magnitude (turbulence intensity), were not modelled correctly.

A recent study of cladding pressures on a proposed building in Hong Kong, which had lower edge discontinuities and very high intermittent negative pressures, prompted the author to look carefully at wind characteristics in tropical cyclones, Melbourne and Blackman [2]. This resulted in a major study of tropical cyclone wind data collected at the Hong Kong Royal Observatory and at Waglan Island, including modelling the position errors for both of these anemometer sites. The conclusion was that in extreme tropical cyclone events the lower part of the atmospheric boundary layer has turbulence characteristics similar to flow expected over suburban roughness (i.e. z_o = 0.20 m). This is contrary to many earlier findings in respect of wind characteristics over the sea. Because these tubulence characteristics are so important to the determination of cladding pressures, it is worth emphasising that the effective surface roughness of the sea in extreme tropical cyclone conditions is much higher than previously thought, and this also is relevant to wind effects on offshore as well as onshore structures.

4.2 Grandstand roofs, bridge decks

A phenomenon allied to that in §4.1 is the wind loading on cantilevered grandstand roofs. Again it is the high levels of incident turbulence through the shear layer re-attachment system which can cause very high response of these roofs, much higher than predicted by quasi-steady codes. This phenomenon was illustrated by Melbourne [3,4] and a method of reducing the wind loading was put forward. This entailed using a slot behind the leading edge which bled flow into the re-attaching shear layer bubble and which in turn prevented the development of very high loads. This suggestion has recently been taken up for a grandstand roof constructed by BASC Contracts Ltd, Cook [5].

The response of large span box girder bridge decks has been shown to be much more dependent on the magnitude of incident turbulence, Melbourne [6], than was predicted using quasi-steady assumptions and strip theory. Full scale and model studies on the cable stayed box girder West Gate Bridge (centre span 336 m) in the City of Melbourne concluded that vertical deck response was increasing approximately with turbulence squared. Again the mechanism is related to flow phenomena near the leading edge which are very dependent on the incident freestream turbulence, which, it is suggested, can be reduced by permitting bleed flow through a porous or slotted leading edge configuration in much the same way as for the grandstand roof.

These examples of the major influence of turbulence and the way in which leading edge pressures in high turbulence flows may be controlled by bleeding flow into the re-attaching shear layer region perhaps have other applications, even to industrial buildings which could be designed with slotted leading edge roof configurations.

4.3 Interference effects

Many authors have instanced examples of where the response of structures in the interference wake flow of other structures has been greatly increased. This has included full scale examples involving buildings, chimney stacks and bundled electrical conductors. With respect to the latter two, work by Wardlaw [7] and Ruscheweyh [8] have indicated just how complex these problems can be with the involvement of many excitation mechanisms. However, with buildings the problem seems much simpler, that is simpler to understand and simpler to predict.

The major enhancement of building response in the wake flow of other stuctures seems to be attributable to the increase of incident turbulence caused by the wake flow. For example, Melbourne and Sharp [9] showed that increase in interference effects was relatively much greater when the buildings were in low surface roughness terrain. In this situation the increase in turbulence level caused by an upstream building is relatively much more than when the original approach turbulence is due to city building type roughness. The danger, in a design sense, is to design for the response of a tall building in a shore front or open country exposure when there is any possibility of similar buildings being built upstream (on reclaimed land in respect of the former situation). A single upstream building in these situations can produce much greater response on a downstream building than if the building were in a generally rougher terrain (more turbulence) because when the interference occurs at the edge of the building wake we have the worst combination of high mean wind speed, due to the small roughness approach terrain, and the high turbulence at the edge of the wake. There is a case for saying that all tall buildings designed initially for open country or shore front exposure should be checked (designed) for both along-wind and cross-wind response with city centre type turbulence levels superimposed on the same open exposure mean wind speeds. Such a procedure would go a long way towards eliminating the interference response problems for tall buildings.

4.4 Reynolds number/turbulence effects

To illustrate that not all increased turbulence effects produce higher wind loading and the effect of Reynolds number, we can consider structures with curved surfaces, in particular circular structures like chimney stacks.

It has been known for some time that predictions of circular chimney cross-wind response based on data obtained from aeroelastic model tests tend to overestimate the full scale experience. Fortunately most reinforced concrete stacks are such that the design is dictated by the along-wind response; only for very large, stiff stacks (such as during the erection phase) will the cross-wind critical wind speed be high enough in the wind speed range for cross-wind response to dominate design considerations.

To quantify the effect of turbulence on circular cylinders over a range of Reynolds number from sub-critical to super-critical, a major study has been undertaken at Monash University. Some of these results have been reported by Cheung and Melbourne [10] and the effects of turbulence can be seen to be acting in two ways. In Figure 3 an example has been given of the variation of fluctuating lift with turbulence intensity and Reynolds number.

At sub-critical Reynolds numbers the effect of increasing turbulence is to decrease the fluctuating lift. As the organised shedding disappears in the critical flow regime, the fluctuating lift drops to a lower value. At super-critical and transcritical Reynolds numbers increasing turbulence causes the fluctuating lift to increase. Similar characteristics are shown with respect to the fluctuating drag component.

Prediction of the response of structures with curved surfaces can be seen to be very difficult, and wind tunnel tests conducted at Reynolds numbers below 2×10^5, even with the correct turbulence, are quite meaningless in terms of providing data relevant to full scale structures.

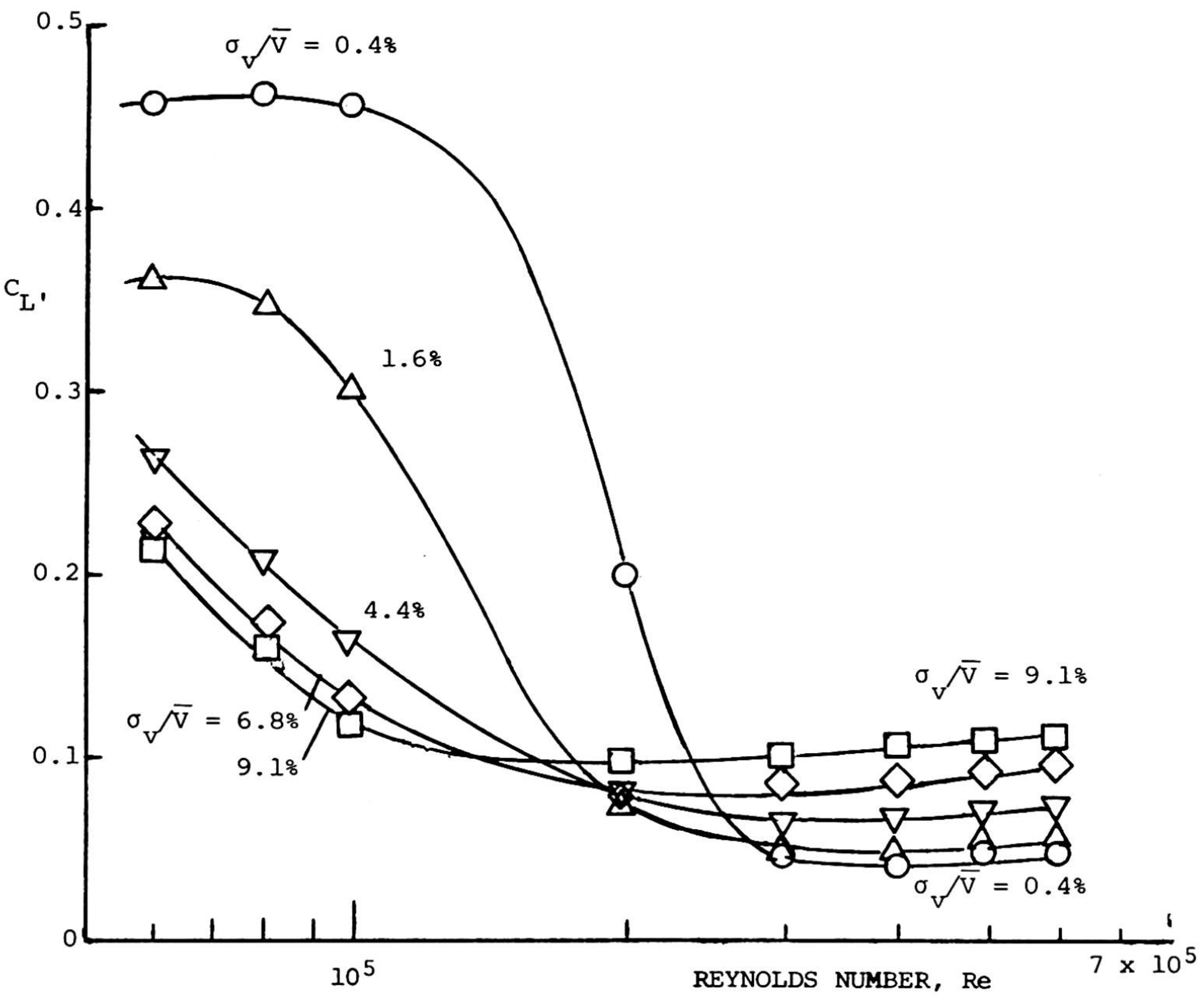

Fig. 3 Fluctuating lift coefficients on a circular cylinder as a function of Reynolds number for different turbulence intensities.

CONCLUSIONS

This introductory paper has discussed two aspects of wind loading on structures which are common knowledge amongst wind engineering researchers and used by some of the leading structural designers. The discussion of inertial load distribution, and the use of design wind speeds with direction has been aimed at exciting interest in bridging the gap between advanced knowledge and general practice by emphasising that both of these real wind loading concepts can be simply incorporated into everyday structural design.

The remainder of the paper has discussed a number of effects of freestream turbulence on the wind loading of structures with the objective of drawing papers dealing with these more complex problems.

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Seminar VII

Developments in the Construction of Steel Structures

Développements dans l'exécution de constructions métalliques

Fortschritte in der Ausführung von Stahltragwerken

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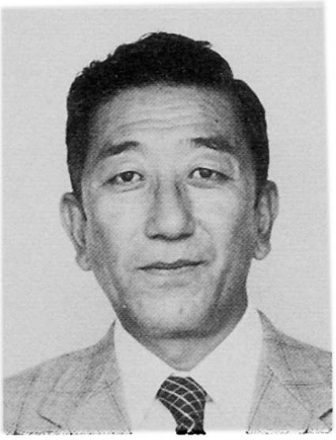

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SUMMARY
Development in steel construction is being supported by various technologies: design process, steel quality, welding technique, quality control in fabrication and method of erection. In this paper, experiences of quality control of high strength steel in manufacturing and advanced column-to-beam connections are reported.

RESUME
Les développements dans l'exécution de constructions métalliques sont dus à diverses technologies: méthodes de calcul, qualités d'acier, technique de soudage, contrôle de la qualité dans la fabrication et méthodes de montage. Le rapport mentionne des expériences du contrôle de la qualité d'aciers à haute résistance lors de la fabrication, ainsi que de nouvelles liaisons poutre-colonne.

ZUSAMMENFASSUNG
Die Fortschritte in der Ausführung von Stahltragwerken werden von verschiedenen Technologien gestützt: Entwurfsprozess, Stahlqualität, Schweisstechnik, Qualitätskontrolle in Fabrikation und Montagemethoden. Dieses Referat berichtet über Erfahrungen der Qualitätskontrolle des Stahles von hoher Festigkeit in der Fabrikation und der modernen Säule-Balken-Verbindung.

1. INTRODUCTION

Iron and steel for structural use have bee developed as shown in Fig.1 from cast iron up to quenched-and-tempered high strength steel during two centuries. In Fig.2 amount of crude steel production in U.S.A., Europe(sum.of U.K., France and West Germany) and Japan are shown. Production of Steel was increased remarkablly after the second World War in Europe and Japan. In the figure, names of suspension bridges, truss bridges and arch bridges are shown together in the order of span length and year completed. Author picked up the bridges as the one sample of steel structure in the figure. The center span length of bridge reached 1280m in 1937. However, it is a interesting fact that span lengths of bridges after the second World War have the same tendency as the increasing of product of crude steel in these three regions. Tendency of increasing of steel product in Europe and Japan are behind ten or twenty years that in U.S.A., and so are bridge span lengths.

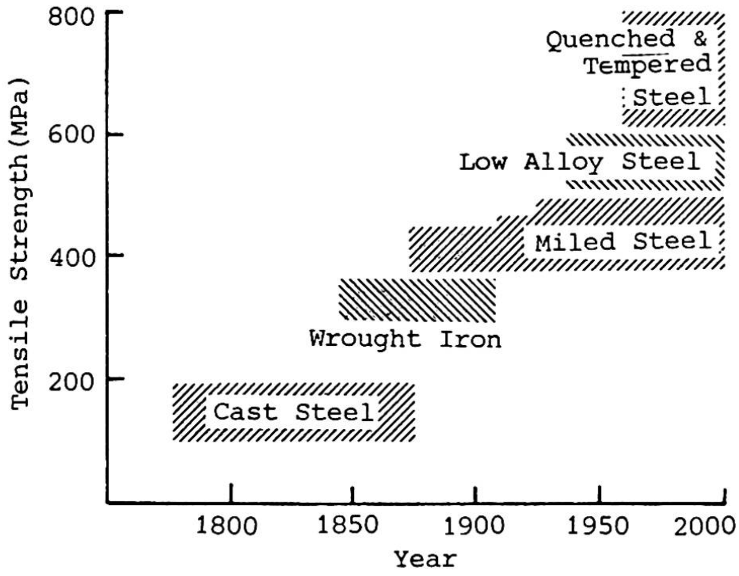


Fig.1 Vicissitude of Structural Steel

In recent years product of steel are hovering round according to the economic situation in the world. Nowadays, big bridge projects like the

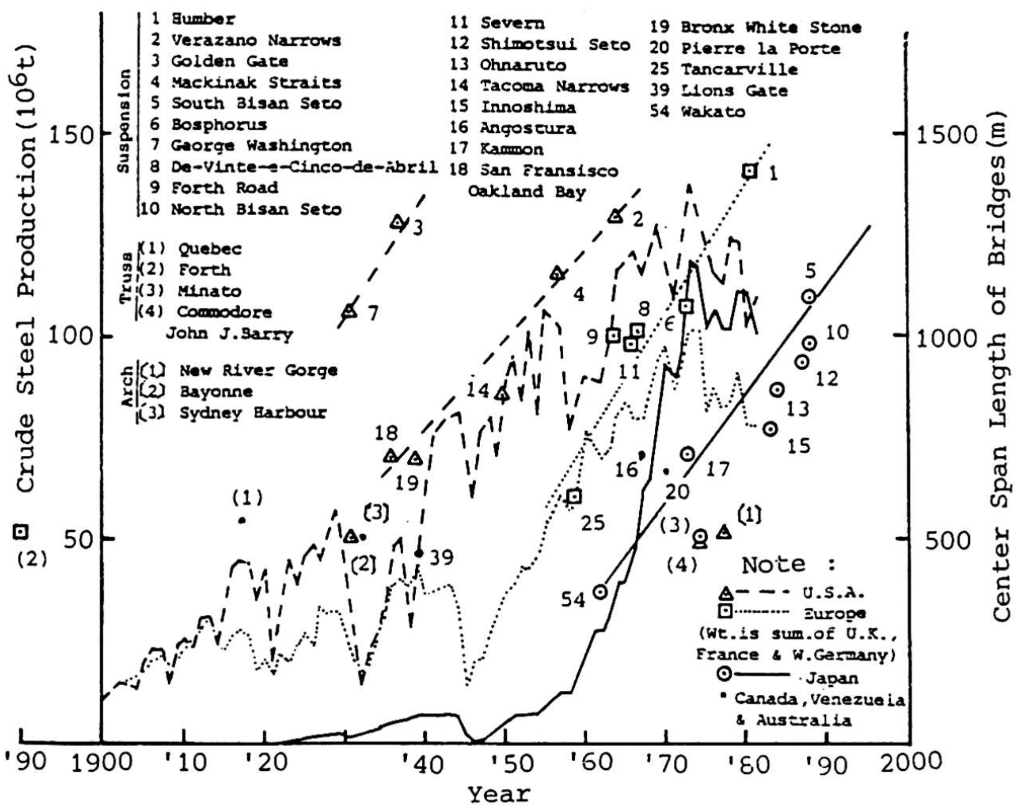


Fig.2 Crude Steel Production and Span Length of Bridges

Akashi-kaikyo Bridge - Japan, 1780m, Great Belt Bridge - Denmark,1416m, Messina Straight Bridge - Italy, 3000m, are postponed also.

Fig.3 shows the increased adoption and development of steel-frame buildings in Japan in the last twenty years. It was in the latter half of the 1960s that steel-frame structures overtook reinforced concrete structures in terms of floor area. Two reasons for this phenomenon are conceivable. One is the natural environment of Japan since it is an earthquake country and lies in the path of typhoons so that steel-frame construction is suited for meeting severe mechanical requirements. The second reason for the marked increase in steel-frame construction is the emergence of a number of a structural forms more readily meeting the need for improved productivity as a result of increased technical information exchanges and joint research and development between various engineering field. In the field of bridge construction, total cost of construction of steel superstructure is about 50% over the prestressed concrete bridges in recent five years in Japan(Fig.4).

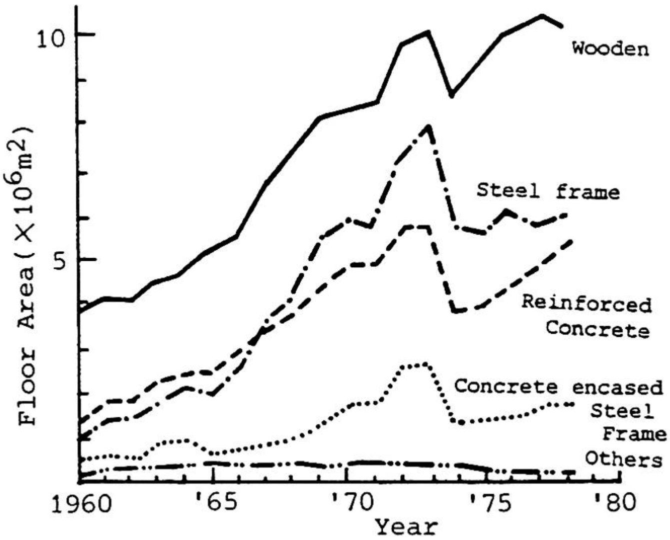


Fig.3 Building Floor Area Constructed in Each Year Classified by the Structural Material

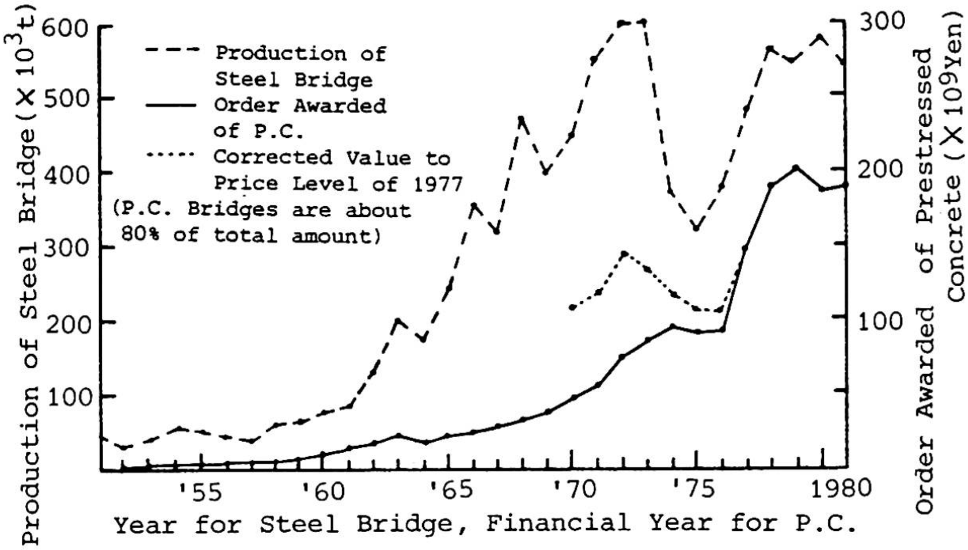


Fig.4 Production of Steel and P.C.Bridges

Development of steel construction is being supported by technologies in various fields such as the progress of design process, steel quality, welding technique, quality control in fabrication and method of erection (Fig.5and 6).

On some steel constructions investigations or problems in some construction stage are given in following examples.

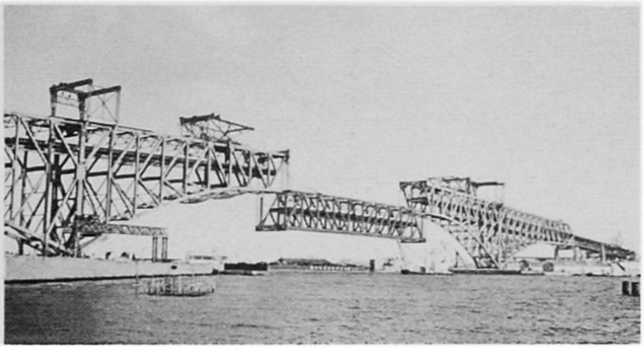


Fig.5 Osaka Port Bridge(1973)
Suspended Span 186m, 4500t

Fig.6 Izuni Otsu Bridge(1976)
Span 172.572m, 3000t

2.APPLICATION OF HIGH-STRENGTH STEEL ON LONG SPAN BRIDGES

Carquinez-Strait Bridge is well Known as the application of Quenched-and-tempered 800MPa class steel which was completed in 1958. De-Vinte e Cinko de Abril Bridge, completed in 1967 over the River Tejo in Lisbon Portugal, is also famous as the long spanned suspension bridge in Europe. This bridge is expected to be able to carry the railway in the second stage of construction, and quenched-and-tempered 800Mpa class steel was used for the stiffening truss members.

Application of quenched-and-tempered high strength steel was started since 1961 in Japan, but at that time of 600MPa class, and standard design specification of bridges made of this class steel was set in 1967 for highway bridge and in 1974 for railway bridge. For highway bridges 600MPa class steel is used widely in Japan nowadays.

700MPa and 800MPa class steels were applied for the first time for the huge bridge - Osaka Port Bridge, cantilever truss with 510m center span length in 1974(1) (See Fig.5). These steels had high weldabity and were newly produced for the construction of this bridge. Before the construction of this bridge, application of 800MPa class steel was tried for a highway plate girder in 1964, and for railway trial application was done in 1971 for center girder of a through type plate girder bridge with three main girder (2) .

Honshu-Shikoku bridge has three route and the total length over the sea is about 30km. It is composed of eleven suspension bridges and other long span cable stayed bridges etc. Then, one harf of the whole project is under construction now. In these bridges four suspension bridges which have the center span length 876m, 940m(Fig.7), 990m and 1100m and two cable stayed bridges(Fig. 8) which have the center span length 420m are highway-railway combined bridge.

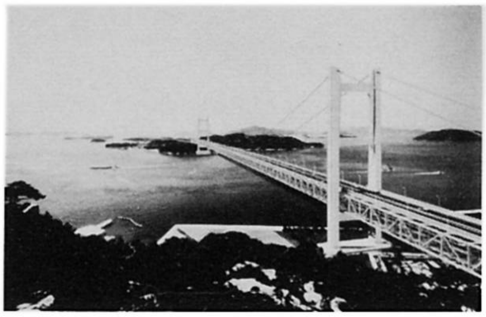


Fig.7 Shimotsui Seto Bridge
Center Span 940m

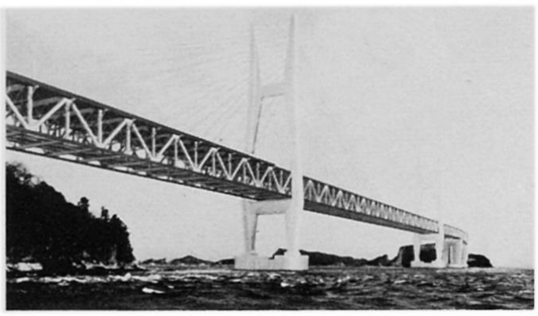


Fig.8 Iwagurojima Bridge
Center Span 420m

The type of suspended structures of these bridges are truss type with box-section members. The dimension of them are large as shown in Fig.9 and quenched-and-tempered 700MPa class and 600MPa class high strength steels are applied. Because of the railway carrying bridges, careful attention must be paid for effect of load repetition in designing and manufacturing of these bridges.

Abreasting with the establishment of fatigue design code [3], many fatigue tests were conducted to certify the fatigue strength with large size specimens [4],[5]. Based on these results design code was revised 1982 [6], and relation between the imperfection of root of weld in the corner welding and fatigue strength was clarified as shown in Fig.10. For the corner weld of box section as shown in Fig.9, partially penetrated groove weld and fillet weld are used and the welding is only one side weld from the outside of box section.

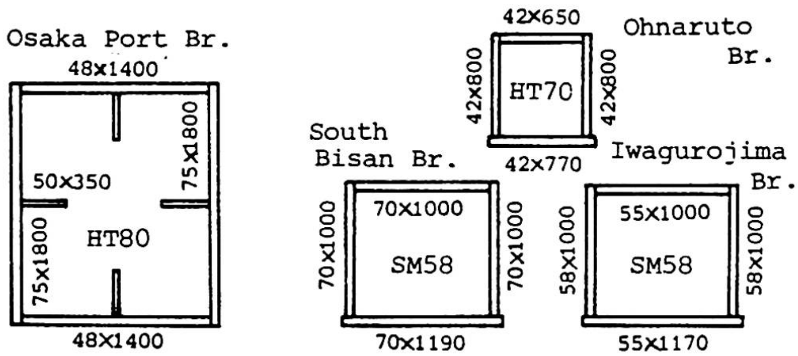


Fig.9 Maximum Section of Chord Member

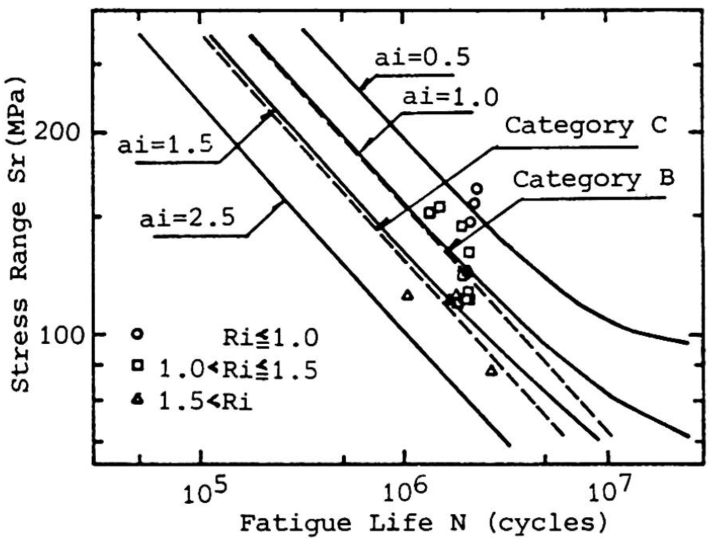


Fig.10 Predicted Sr-Np Curves and Test Results

Discontinuity of root of weld - blow holes, slag inclusion, drooping of weld metal and irregularity of root line - must be controled in every respect of above mentioned conditions. In the fatigue design initial size of imperfection at the root of weld was estimated as ai=1.0mm.

To keep the quality of welding reliable, many procedure test has been conducted with small L type specimens, specimens of full size section with medium length and full length of chord member which had the length of 20m. From the results of these tests quality control points and standard process of workmanship were established in every stage of fabrication . Main control points are cleaning of surface for weld, keeping the root opening less than 0.5mm(Fig.11), putting the sealing bead at the root and selection of proper welding conditions. Some solutions for them are as follows:

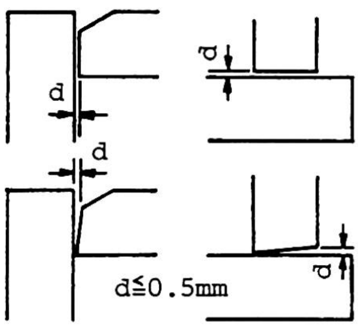


Fig.11 Tolerance of Root Opening

1) Plates used must be flat. In the standard of Honshu-Shikoku Bridge Authority(HBS G 3103), flatness of steel Plate is specified as less than 2mm measured in 1 meter span for 600MPa class steel over 50mm thickness and for 700MPa and 800Mpa class steel over 32mm thickness.Flatness of plates with the thickness less than these values is supposed to follow in the tolerance specified in JIS.

However, before and after the cutting and welding the plates correction of small waving are conducted for every plate.

2) Plates must be cut straight and to be square in its edges. After cutting plate edges are finished with automatical running belt grinder or machined. No drag lines due to flame cutting must be remained.

3) After the tack weld, sealing beads must be put between the tack welds in full length by semi-automatic CO_2 welding or MAG. Size of these welding must be nearly 4~5mm because they can be remelted by the corner welding conducted submerge arc welding.

4) Examples of welding condition in the Ohnaruto Bridge using 600MPa and 700 MPa class steels are shown in Table 1.

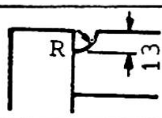
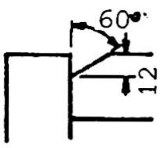
	Shapes of Groove	·Kind of Weld	Welding Condition of 1st pass						Welding Material
			Current (A)		Voltave (V)		Velocity (cm/min)		
A		Single S.A.W.		600		30		30	Wire: dia. 4.0 $\sim 4.8^{mm}$ Flux: Fused Type
B		Single S.A.W.		750		32		26	
C		Tandem S.A.W.	Front	700 ~750	Front	30 ~33	HT 70	60	
			Rear	550 ~ 620	Rear	32 ~38	SM 58	55 ~40	

Table.1 Welding Condition

5) Weld lines must be preheated by electric heater up to proper temperature symmetrically preventing the irregular deformation of members. State of the welding is shown in Fig.12.

6) For the check of soundness of these welding ultra sonic wave testing system was newly developed. This equipment runs automatically, and its velocity is 30~50cm/min. Fig.13 shows the system of A.U.T. and state of the testing is shown in Fig.14.

To satisfy the required quality of welding and reap the full advantage of high strength steel, quality control over the workmanship is being conducted not only in the process above mentioned but in every stage of fabrication.

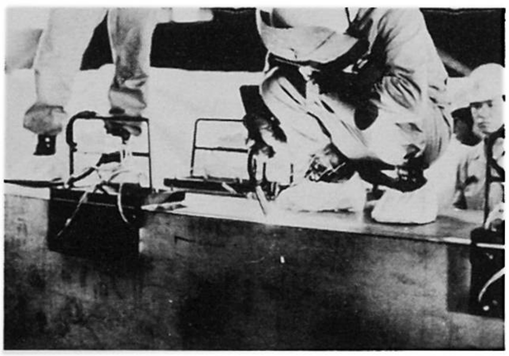
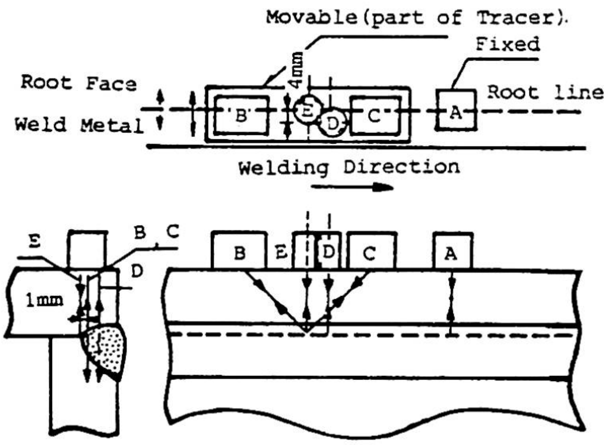


Fig.12 State of the Welding for Sealing



A : Line Focus Type.
Check for Deapth of Penetration
B, C: Point Focus Type.
Check for imperfection of weld root
D : Point Focus Type.
Check for pipe shaped blow hole and imperfection in the weld metal
E : Point Focus Type.
Sensor for root tracer

fig.13 A.U.T.System

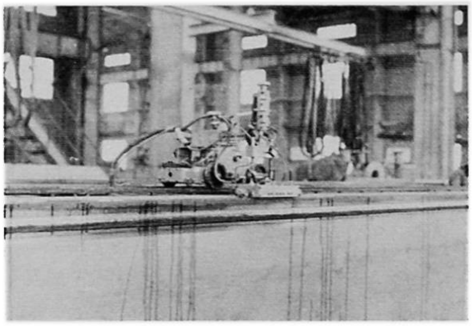


Fig.14 Inspection by U.T

3. ADVANCED COLUMN-TO-BEAM CONNECTION OF STEEL FRAMEWORK

High earthquake resistance and high wind resistance are prerequisites for building in Japan. Meanwhile, whatever such special requirements there may be, a world-wide trend exists aiming for economical buildings erected in short periods of time while maintaining specific levels of quality. For this purpose, structural steel which is comparatively expensive must be used effectively in building production achieving the goal of improvement in safety, ease of work execution and economy of the building as a whole.

In order to increase earthquake resistance and wind resistance performances of frameworks of high-rise buildings as in Japan, it will be effective to utilize wall. However, such performances can also be attained by rigid connection between columns and beams [7] .

The steel framework is to be constructed by connecting the columns and beams successively, therefore "connecting" is the basic means for the production of steel framework. Meanwhile the internal stresses of frame are transmitted via the connection. It means that the connection is the most important element of the steel framework, that could control the safety of the whole structural system. As mentioned above, the connection between column and beam holds the key for the solution of the productivity and the safety of the structural steel framework.

The types of rigid connections between columns and beams normally used in Japan are shown in Fig.15. As indicated in (a), (b) and (c) of Fig.15, the method of welding beam ends to columns is mechanically effective for rigid connections.

However, this method requires a high technical skill including control to secure the reliability of welding.

In case of the field welding, the quality control of welding is vital, where manual arc welding, non-gas shielded arc welding and semi-automatic gas shielded arc welding are used almost exclusively for column-to-beam connection. Meanwhile narrow-gap automatic welding is used at times for the column-to-column field connection where the plate of column is relatively thick. To confirm the soundness of these welding, standard of ultrasonic manual testing for structural joint was prepared by Architectural Institute of Japan in 1973 [8] .

There was another problem of lamellar tearing for structural steel at the welded T-joint connecting beam flange plate to column several years ago. But it is under control by the development of anti-lamellar tearing steel [9] and carefully controled preciseness of steel fabrication.

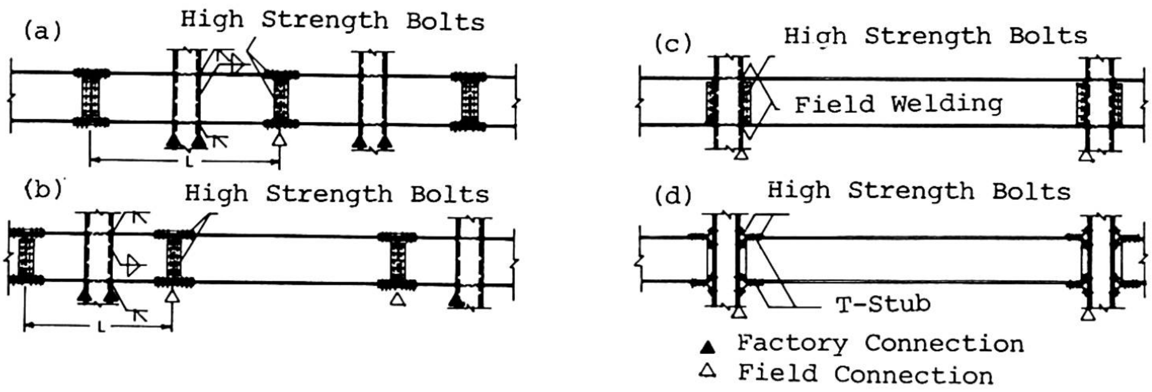


Fig.15 Four Types of Beam-to-Column Rigid Connections [10]

The rigid connection detail shown as (d) was newly developed putting a great improvements into the existing Split-Tee connection and called HISPLIT. This system is a method of simplifying the constitution of the beam-to-column connection while maintaining mechanical performance. The method is to assemble joints by simple tightening of bolts at the jobsite using connection parts mass-produced at the factory. It could be said that the function of the connection is concentrated into this connection parts(Fig.16). Since its development in 1973, this system has been used not only in Japan, but also in the U.S.A., Germany and the Near and Middle East. The connection part is made of cast steel or is stamp-forged. The configuration of the connection part minimizes the lever action occured in the Split-Tee connection system and at the same time allows the internal stresses to be transferred smoothly. Bending moment and shear force must be trnsferred from beam to column through the T-stub web alone. According to the von Mises'yield criterion, the yield stress of the T-stub web when bending moment M and shear force Q act simultaneously can be expressed as,

$$\sigma_y = \sqrt{\sigma^2 + 3\tau^2} = \sqrt{\left(\frac{M}{dA_w}\right)^2 + 3\left(\frac{1.5Q}{2A_w}\right)^2}$$

in which, d = distance between the centers of two T-stub webs
A_w= cross area of T-stub web

The yield strength of the T-stub web based on the formula shown above usually comes about twice as high as the full strength of beam members, even if the local bending in the T-stub web is considered. When the shear arm ratio M/Qd is less than 2, however, we must examine the effect of local bending which may reduce the strength of the connection.

The mechanical properties and shape of T-stub are shown in Table 2 and Fig.17 respectively.

A great amount of the fundamental experiments were carried out during this technology development and moreover the full-size strength tests of this conetion system were conducted to confirm the safety.

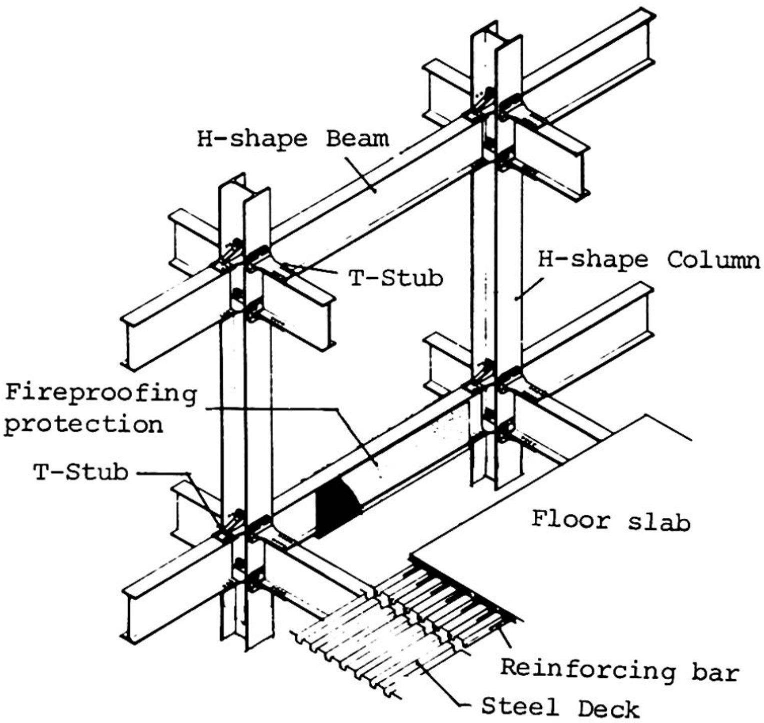


Fig.16 Sketch of Steel framework

Tensile strength	Not less than 490MPa
Yield Point	Not less than 280MPa
Elongation(%)	Not less than 23
Impact value	Not less than $28N \cdot m/cm^2$
Bending 180°	Good

Table 2 Mechanical Properties of T-Stub

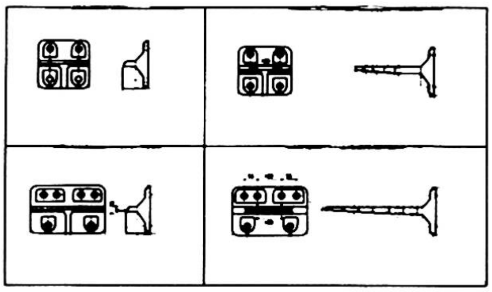


Fig.17 Example of T-Stub

As the practical example of this system, the Akasaka Prince Hotel that was completed in May 1983 is summarized here[12] (Table 3 and Fig.18,19). This hotel is located at Akasaka that is in the center portion of metropolitan Tokyo. The most attractive feature of this hotel is the V-shaped building plan consisting of two wings and this plan is reflected conspicuously in the exterior elevation which is the essence of modern architecture(Fig.20). Meanwhile,the unusual shape of this plan posed some perplexing problems which the structural engineer had to deal with. Two of the problems are described briefly below.

To design a structural system enabling control to minimize stresses and deflections of the building structure against earthquake and wind forces in all directions --- to ensure high degrees of safety during earthquake and storms and not to spoil residential comfort even under such circumstances.

To develop a construction method for the steel framework to simplify the connections of this complicated structural framing in three directions --- to raise productivity of fabrication at the factory, to simplify erection at the jobsite and to ensure a high degree of quality control with minimum efforts. The solutions arrived at are descrived below.

The structural system employed in this building is a combination of a rigid steel framework (4.0m bay x 4.0m bay) and precast concrete shear walls (slitted shear walls) located at the central core portion and at the end of each wing to minimize the torsional deflection of the building. The slitted shear

Architect:	Kenzo Tange + URTEC
Structural Engineer:	Architectural Design Division, Kajima Corporation
Contractor:	Kajima Corporation
Construction Period:	Approx, 3 years
General Description of the Building	
Typical Floor Area:	1,485m^2
Total Floor Area:	67,485m^2
Number of Guest Room:	761
Stories:	39 stories above ground, 2 stories underground
Building Height:	139.8m above ground level
Typical Story Height:	3.2m
Structural System:	
2nd Basement Flr.:	Reinforced concrete structure
1st Basement Flr. - 2nd Flr.:	Reinforced concrete encased steel structure
3rd Flr. and above:	Steel structure by HISPLIT System
Exterior Finish:	Aluminum curtain wall with mirror glass
Structural Materials:	
Concrete:	Approx. 25,000m^3
Reinforcing Steel:	Approx. 2,800ton
Structural Steel:	Approx. 8,500ton
T-Stub:	Approx. 500ton

Table 3 The summary of the Akasaka Prince Hotel

walls mentioned above, in the case of this building, bear 30 to 50% of holizontal forces due to earthquakes and storms. The safety of this structural system against earthquakes or storms was studied and comfirmed in detail by means of a three-dimensional elastoplastic analysis program developed for this project [12].

Steel fabrication at the factory was reduced to a minimum using the simplified connection details, and at the same time erection speed at the jobsite was raised drastically.

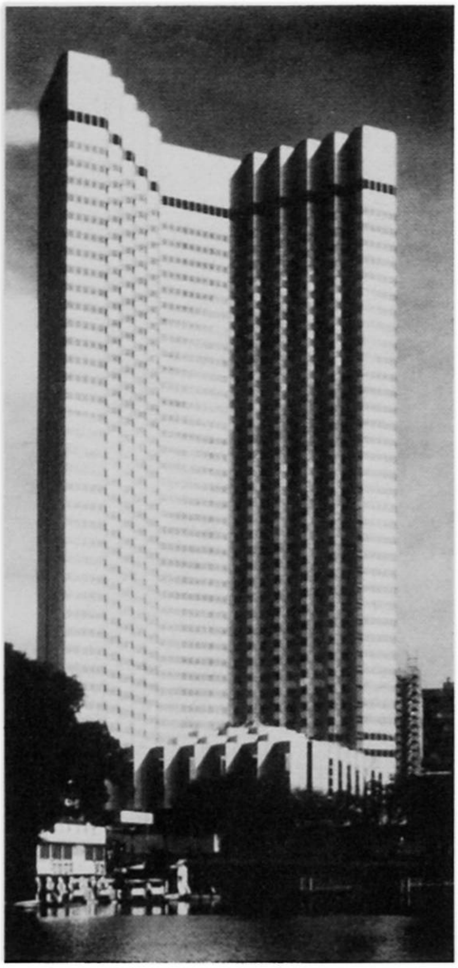


Fig.18 The Akasaka Prince Hotel

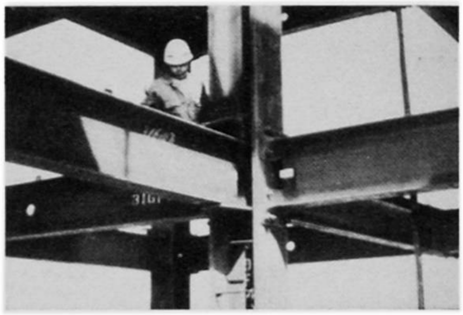


Fig.19 Detail of Column-to Beam Connection

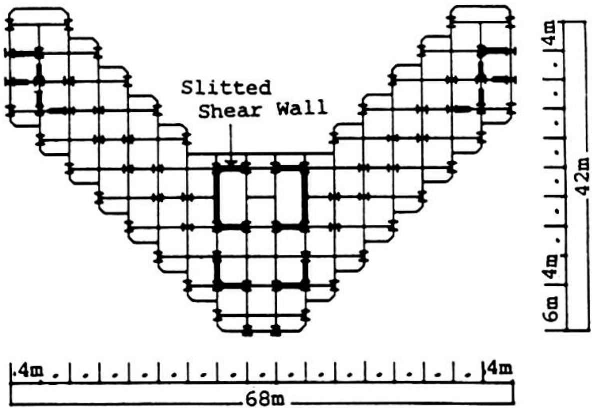


Fig.20 Typical Framing Plan

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Seminar VIII

Snow and Ice Effects on Structures

Effets de la neige et de la glace sur les structures

Wirkung von Schnee und Eis auf Tragwerke

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Chairman
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SUMMARY
This paper outlines the effects of snow and ice conditions on buildings and other structures as experienced by a structural engineer practising in Canada. Both direct and indirect effects are discussed in general and in detailed terms in the light of past and current knowledge and experience. The paper also includes conclusions and recommendations for further research and development to achieve a safer and more cost effective design.

RESUME
Le rapport décrit les effets de la neige et de la glace sur les bâtiments et d'autres structures au Canada. Les effets directs et indirects sont envisagés à la lumière des expériences passées et présentes. L'article présente des conclusions et des recommandations pour des recherches et développements futurs, afin d'obtenir un projet plus sûr et plus économique.

ZUSAMMENFASSUNG
Der Bericht beschreibt die Wirkungen von Schnee und Eis auf Hochbauten und andere Tragwerke, wie sie von einem Bauingenieur in Kanada beobachtet werden. Direkte und indirekte Wirkungen werden aufgrund von Erfahrungen und neueren Erkenntnissen besprochen. Der Bericht enthält Schlussfolgerungen und Empfehlungen für notwendige Schritte in Forschung und Entwicklung, um Sicherheit und Wirtschaftlichkeit des Bauens weiter zu steigern.

1. INTRODUCTION

Buildings located in those intemperate parts of the world, where temperatures can remain at sub freezing levels for lengthy periods, must be designed to withstand the loads and other effects imposed on them by snow and ice. Structural designers who practise their profession in such areas must reach some understanding of the characteristics of these materials and, equally important, the climatological elements which produce them. The term, "some understanding", is appropriate because much remains to be appreciated about snow and ice and their effect on structures exposed to their action.

2. GENERAL DISCUSSION AND REVIEW

2.1 Physical Properties of Snow and Ice

Studies of snow have shown that its physical properties and mode of accumulation are dependent on many variables. Age, moisture content, temperature, exposure to wind, characteristics of the surface on which it falls, are some of the conditions which must be assessed by the designer in choosing the design loadings.

The weight of ice is, of course, known quite accurately. What is not known, is the rate of transformation of snow to ice in a given location on a given surface. Hence difficulty is experienced in determining design snow/ice loads. Past and present experience provides ample evidence that, under certain conditions, the accumulation of snow and ice can cause the failure of structures of all descriptions.

The challenge remains for engineers and designers to increase their knowledge of snow and ice so that the risk of structural failure will be reduced.

Because of their nature, snow and ice loads can be controlled by simply raising their temperature, (or lowering their freezing point), until they melt. Then, with proper drainage, potentially dangerous accumulations simply flow off the structure. Unfortunately, the means used to raise the temperature above the melting point can themselves cause distress to the structure to the point where the effect of eliminating the weight can lead to the very failure which caused concern in the first place.

It is important also, to appreciate the effects of snow and ice on the earth which supports or is retained by a structure. Failure of foundations and walls to maintain the equilibrium of structures as a result of freezing or thawing or repeated cycles of both in the supporting soil, can result in just as significant damage as the failure of spanning members to support gravity loads.

2.2 Influence of Wind on Snow Accumulation

The accumulation of snow is influenced significantly by the wind conditions which are present during its deposition. Indeed it is often the uneven distribution of snow resulting from wind action, which causes the overload conditions so damaging to many structures.

Just as the wind can model and shape sand dunes on beaches and deserts, so can snow surfaces undulate across otherwise flat planes. When roughness is introduced in the form of obstructions to the normal wind flow, the pressure variations affecting the buoyancy of the flow are multiplied in magnitude. Thus the wind can cause snow to be deposited in very large quantities in the lee of obstructions and scour snow from the surfaces exposed to windward. As wind eddies move across roof areas, snow is deposited in low pressure locations thus creating a natural obstruction which creates "lee conditions" over a larger area thus causing the initial drift to grow, so long as the wind bearing the snow continues to blow.

In a similar manner drifting can occur on the lee side of sloping and curved surfaces and wherever higher structures adjoin a lower roof.

Thus, the immediate effect of snow on structures is that of superimposing load on them. The responsibility of the designer is simply to identify the magnitude of the load and its distribution.

2.3 Direct and Indirect Effects of Ice

Like snow, the direct effect of ice accumulations on a structure can be related to the gravity and wind forces resulting from measurable (or forecastable) quantities of ice present. In addition to its greater weight, ice, due to its bonding characteristics can coat exposed structural elements to such a degree that their cross-sections are magnified drastically thus exposing them to much higher wind force as well as weight. Bridges, antenna structures, power lines and similar types of structures must, therefore, be checked for the greater surface areas exposed to wind than that presented by the structural members themselves.

If snow's principle effect is to load structures, the effects of ice are at once, more far reaching and insidious. The principle and most difficult problems with ice are related to its transient, physical qualities. In all but the coldest regions of the world, ice and snow are subjected to recurrent periods of alternating freezing and thawing conditions. During thawing conditions, the melt water is free to infiltrate structures. If, having penetrated the fabric of a structure, conditions change so that ice again forms, the expansive forces thus generated, place the very marrow of the structure at peril. If subjected to sufficient repetitions, the structure's useful function will end. No common structural material - - steel, concrete, or timber, can successfully resist the effect of repeated freezing and thawing of water which has penetrated its surface skin.

2.4 General Summary

Thus, in general terms, the effect of snow and ice on structures can be related to:

2.4.1
The gravity forces imposed by their weight and the weight of equipment used to remove them,

2.4.2
Wind forces acting on the structure as a result of enlarged areas due to the build up of ice,

2.4.3
Internal disintegrating forces caused by water which has penetrated structural materials changing to ice,

2.4.4
Disintegrating forces caused by corrosive salts in solution with such penetrant solutions causing the oxidation of structural materials;

2.4.5
Heaving/settling movements of foundations due to freeze/thaw of supporting soil.

3. DETAILED DISCUSSION

3.1 Gravity Effects

The phenomenon of snow drifting in the lee of obstructions is generally recognized. Building authorities throughout the colder regions of the world require provisions to be made for the phenomenon in the form of higher loads on structural areas adjacent to such obstructions. The undulating surface of snow is also recognized by such authorites by requiring structural members supporting such areas to be designed to resist both uniform and non-uniform distributions of snow loading. Drift effects are also recognized on the lee side of pitched roofs and again both uniform and non-uniform distributions of snow are required for the design of such roofs.

The current (1980) issue of the National Building Code to Canada outlines in considerable detail the provisions which must be made for uniform and non-uniform snow loading on buildings. These requirements are supplemented by considerable discussion on snow loads in a commentary contained in a supplement issued with the actual Code itself.

The simple identification of locations where snow drifting is likely to occur and the likely drift depth can, at best, provide only qualitative information for snow accumulation. The climatological history at the site of a given structure must be considered before quantitative values for the accumulation of snow over the winter season can be considered.

The quantitative values for snow loading provided by codes like the National Building Code of Canada are based on observations and measurements which have been made for many years. Most other Codes for areas subject to significant snow accumulations, also base their requirements on field observations in appropriate locations. Countries such as Canada which are large and which have extremely varied climatology, can, at best, establish design snow loads based on the results of observations at relatively few locations, to represent quite large areas. A balance must be struck between dangerous underestimation of likely snow and ice accumulations and the economic losses resulting from overly conservative estimates. Designers of buildings and other structures exposed to snow loadings must, in every way possible, encourage those engaged in snow research, in their attempts to quantify more accurately, the design loads which should be used. The frequent news of structural failure resulting from excessive snow loading bears witness to the need for such continuing research.

The density of snow increases with its age to values approaching that of ice. This factor must be recognized by the designer when dealing with structures located in areas unlikely to experience thawing conditions over long periods of time. Also, roof structures with varying insulating characteristics, can experience critical accumulations of ice loading caused by melt water from snow lying on poorly insulated roof areas suffering high heat loss from within, draining and refreezing on unheated eaves or other areas where little or no heat is present.

Often the designer must look beyond the simple identification of the design snow load which can accumulate. In many cases, the weight of equipment used to dispose of or otherwise remove the snow, can be a critical force on the structure itself. On bridges, snow plows can literally pile the snow into drifts at the sides of the road bed itself. Plazas over underground parking garages, often must be recognized as being exposed to the weight of heavy snow removal equipment which can gain access to the plaza area from adjacent roadways. Owners of open air parking garages often may create signicant overloading by designating a storage area within the garage itself to pile snow removed from the other garage areas.

3.2 Wind Effects On Ice Coated Structures

While the effect of ice storms does not significantly increase the overall shape of buildings exposed to wind forces, the very opposite can be true of slender structures such as antenna towers and transmission towers. These structures are often designed to employ relatively slender, individual elements of small dimension to make up the skeleton trusswork which forms the tower. It is obvious that a truss component 10 cm in width, when exposed to an ice storm capable of causing a 3 cm build up of ice, will result in a drastically increased obstruction width to wind forces. It is important to remember also, that the weight of such coatings can result in significant additional P-delta forces as lateral sway due to wind on the increased obstruction area occurs.

3.3 Effects of Internal Forces Caused By Freezing

The expansive forces which result from the freezing of water which has penetrated the structural elements themselves is one of the two most common causes of structural distress.

While rain and melting snow are obvious sources of external water available to penetrate structural elements, condensation of water vapour escaping from the interior of building structures comprises a third major source of "penetrant" water. External water can be prevented from entering structures by designing or otherwise protecting the integrity of their external surfaces against cracks or other flaws allowing the ingress of water. Condensation water on the other hand requires the installation of efficient vapour barriers on the warm inside surfaces to prevent the passage of water vapour into the structural element. Energy conservation measures in the form of increased insulation of structures due to higher energy costs have disclosed the weakness of vapour barrier design previously thought to be adequate. This is particularly true in older structures. In fact, in a few months, structures which had existed happily for 100 years or more prior to the installation of additional insulation materials, have suffered phenomenal deterioration as a consequence of condensation caused by faulty air and vapour barriers.

Water, which has penetrated the actual structural element or material and then freezing, has great potential to cause damage. The expansive forces resulting from the freezing of internal water in concrete and masonry materials cause the matrix of the material to be broken up. Repeated freeze/thaw cycles can lead to the complete disintegration of the material. Similarly, the formation of ice in joints or cracks in a structural element increases their width and, in the case of internal cracks, causes spalling of the surface. The damage resulting from these (crack widening) effects, accelerates because the widening/spalling/disintegrating conditions allow increased quantities of water to penetrate with a consequent increase in damage.

The entrapment of water within steel members can cause extensive damage if allowed to freeze. Hollow Structural Sections have been seriously damaged by the freezing of entrapped water due to the failure of steel detailers and fabricators to provide drainage holes.

3.4 Effects of Corrosion By Melt Water

Linked with the damage done by water penetrating the fabric of the structure and freezing, and possibly still more destructive, is the internal and external corrosion caused by the acidic nature of solutions resulting from the use of de-icing chemicals. In North America, calcium chloride is commonly used to melt ice and snow on northern and high altitude highways. The useful life of structures exposed to the effect of such chemical ice and snow control methods can be shortened significantly unless expensive and continuing maintenance and repair procedures are adopted. Roadway bridges are the obvious victims of such corrosion and evidence of corrosion damage caused by de-icing road salts can be found in practically any cold climate area of this continent.

In concrete structures the attack commences with the corrosion of embedded, steel reinforcement and pre-stressing tendons. The resultant oxidation produces expansive forces greater than the concrete cover can resist, thus producing cracking and spalling which, in turn, allows increased quantities of acidic solution to penetrate the member. Structural steel girders and other members are also affected by similar action, particularly where dust and dirt can collect to form a poultice which remains damp and in continuing contact with the steel surface.

Although the exposure may not be so great, parking garages, both indoor and outdoor varieties, have been found to suffer in a similar manner to bridges. In this case the acidic solution is transported to the floor surfaces of the garage by the vehicle's tyres and by ice and snow which have collected on the automobile. Because such structures are usually designed under the provision of Building rather than Bridge Codes, the effect of corrosion often becomes critical earlier in the life of the structure. Parking garage structures, particularly those for automobiles, are generally much lighter than bridge structures, due to the relatively light loading and modest depth of cover for reinforcement required by many codes. Melt water therefore, need penetrate a shorter distance to come in contact with reinforcing or pre-stressing tendons. Because the reinforcement tends to be relatively small in cross-section due to the light loading, the reduction in effective area due to even modest corrosion can, in fact, represent a significant percentage of the total cross-sectional area. The cost of repairing damage, and maintenance in general, can be very high.

3.5 Effects of Frost in the Soil

Structures located in cold climate areas must also take into account the presence of ice (frost) in the soil. The expansion forces created when water freezes in the ground can impose quite irresistible lifting forces on structural foundations and walls. By the same token, foundations placed on frozen ground can suffer serious settlement if the frozen supporting soil is allowed to thaw. In both cases cracking and damage to the superstructure can result from such foundation movement.

Care must be taken by the designer to identify locations around structures where wind scour or snow-clearing will prevent the normal accumulation of snow. Deprived of this (snow) insulation, frost can penetrate much deeper than normal.

It has been established that ground frost can develop very significant bond to the vertical surfaces of foundations such as pilings and walls. Under some conditions of repetitive freeze-thaw, this bond strength is sufficient to cause very significant lifting and movement of driven piles and foundation walls, particularly those constructed of jointed masonry. In those areas where the climate is sufficiently cold that the ground remains either completely or partly frozen throughout the entire year, this effect of ground ice must be carefully considered.

4. CONCLUSIONS

This dissertation has attempted to identify in a general way, the various effects which ice and snow have on structures. As stated, some of the effects are direct - others are secondary in the sense that they result from attempts to control the direct effects.

Not all of the control methods have deleterious effects on structures. Designers have developed numerous methods for minimizing ice and snow effects and producing cost effective, relatively maintenance free structures. As stated earlier, continuing study and research, including model testing in wind tunnels and water flumes, has allowed quite accurate forecasting of snow accumulation tendencies for a given structure in a specific location. As a result designers can adjust buildings' shapes and orientation to minimize the effect of snow drifting. The improvement in electrical control systems has allowed the integration of heating systems and insulation to control the build up of snow and ice at critical times, without invoking high costs when less critical conditions prevail.

Designers also have learned more fully, the response of structures to the various influences imposed on them by their environment. Of primary importance in this regard is the appreciation of the effect of structural movements resulting from temperature changes, foundation settlement, shrinkage and creep. This awareness of structural movement has led to great improvement in crack management and control. As a result designers can forecast in a rational way, the optimum location of control joints and the likely movements which will occur at them. This knowledge, together with the improved sealants available today, leads to a marked reduction in the entry points available for melt water to penetrate the skin of the structure, thus causing a marked reduction in potential damage due to corrosive and icing effects.

Much knowledge has been gained also in the control of air and water vapour. Modern materials and methods have been, and are being, developed to prevent more efficiently, the flow of interior air and water vapour through the interior surfaces of structures. Waterproof membranes and surface coatings have been, and are being, developed to protect the surfaces of structural materials against penetration by melt water resulting from ice and snow melting activities.

Despite improved performance, really dramatic advances are dependent upon continued and increased research. The present data base for snow information depends on tedious observation methods conducted by weather stations and other agencies at various locations. In Canada, current and past observations have been confined to measurements of ground snow and to some extent snow accumulation on roofs. The funding and actual snow measurement operations almost always are provided by and through Governmental Agencies. Possibly because of the non-dramatic nature of the subject, appropriate funds and assignments have rarely been provided. Information on snow accumulation is necessary in many more locations than are presently studied. Increased staffing by qualified personnel is required to assist in the measurement, and more importantly, in the recording and assessment of data coming from field observations. It is recognized that the allocation of funds for this work in economically depressed times is difficult to obtain. But the savings resulting from the reduction in structural damage caused by the direct and indirect effects of snow and ice, certainly would justify this expenditure.

Such studies would allow designers to improve their techniques in providing structural resistance to the effects of snow and ice. With this increased knowledge the provisions in Building Codes and other mandatory requirements could be modified to allow both the need for increased resistance in critical areas while at the same time allowing relaxation of requirements in other locations where the effects have been found to have no significance.

5. RECOMMENDATIONS

The persistent occurrences of structural failures, sometimes catastrophic, of buildings and structures due to snow and ice accumulations on their surfaces; the increasing maintenance and repair costs of bridges due to the popular demand by motorists for snow and ice-free roadways throughout the winter; the sometimes rapid deterioration of old buildings subjected to well intentioned, but ill conceived, energy conservation upgrading; the ever present temptation, for economic reasons, to reduce structural member sizes in accordance with increasingly accurate and reliable analysis and testing techniques; - - all these provide justification for the following recommendations.

5.1

Funds and personnel should be made available to expand the data on snow accumulations on the ground and on roofs, through increased field observations and accelerated data review and assessment activities.

5.2

Research relating the influence of the climatology (wind, precipitation, temperature) on snow accumulations in a given location and leading to statistical probabilities of snow drift loading occurring in specific areas on and around a structure, would allow significant cost savings to be realized. With such information, strength could be added efficiently in the proper locations without incurring the cost of strength in locations where there is little or no probability that heavy concentrations of snow can occur.

5.3

Development of wind tunnel testing techniques such that the action of snow and possibly ice accumulations on structures can be accurately scaled so that quantitative snow loadings can be determined in accordance with the climatology of the site in a manner similar to that which can already be obtained for wind loading.

5.4

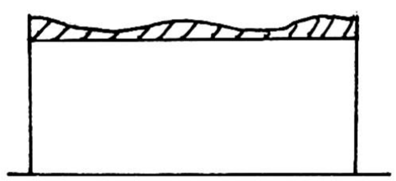
Continued research on protective measures which may be employed to combat the effects of corrosion and the formation of ice on and beneath the surface of structures and structural materials.

5.5

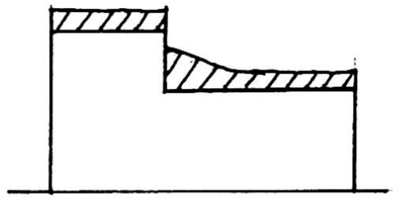
Development of ice and snow removal, chemicals and methods which do not produce by-products which are actively corrosive to structural materials.

5.6

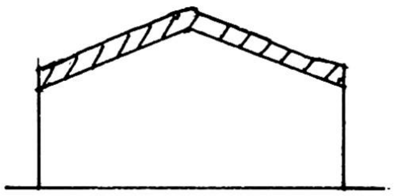
Finally, and possibly most important, an increased international effort to integrate the knowledge and research information presently available only from the research and technical departments of individual Governments and private industries, so that methods to assess and control, more accurately and efficiently, the effects of snow and ice on structures can be universally developed.



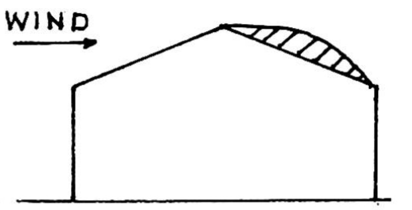
FLAT ROOF UNDULATING SNOW LOADING



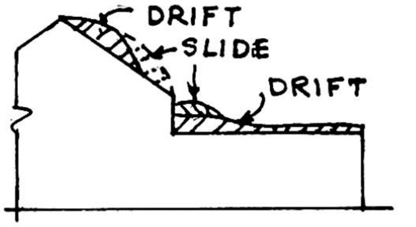
DRIFT ON LOWER ROOF



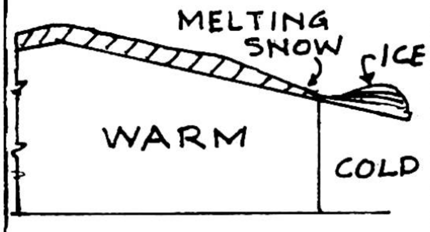
UNIFORM SNOW LOAD (NO WIND)



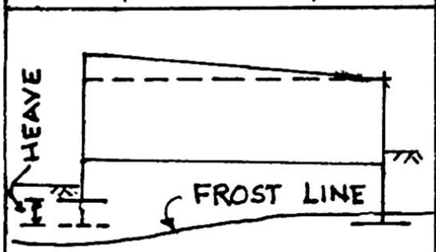
UNBALANCED SNOW (ON LEE SIDE)



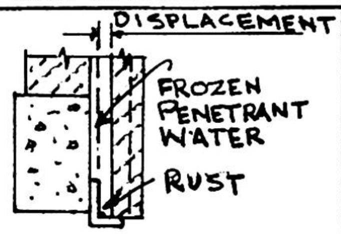
LEE SIDE DRIFTING SNOW SLIDING



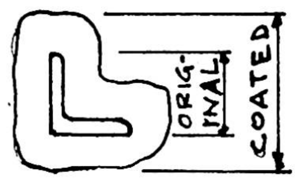
MELT WATER FREEZING ON COLD EAVE



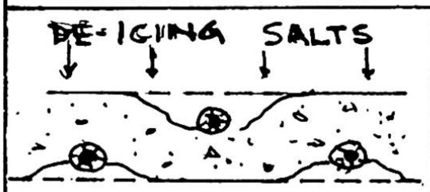
ICE HEAVE ON FOOTING



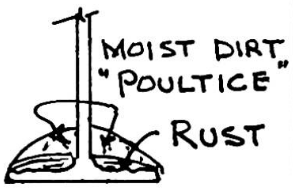
DISPLACEMENT OF FACE BY ICE AND CORROSION



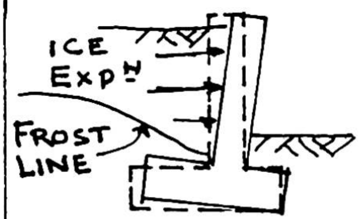
ICE COATING - INCREASED OBSTRUCTION TO WIND



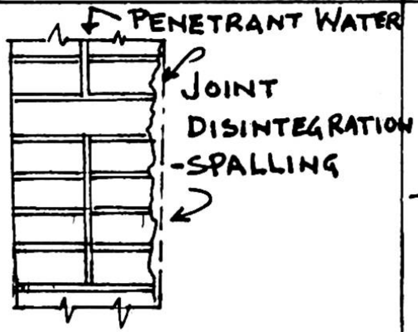
CONCRETE SPALLS DUE TO REBAR CORROSION



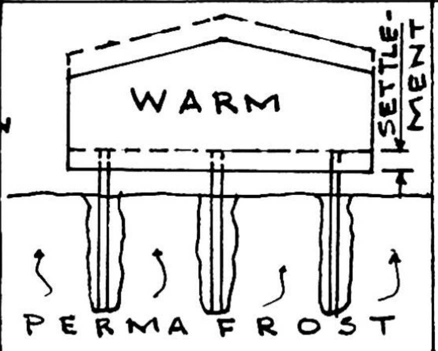
ACTION OF DE-ICING SALTS ON STRUCTURAL STEEL



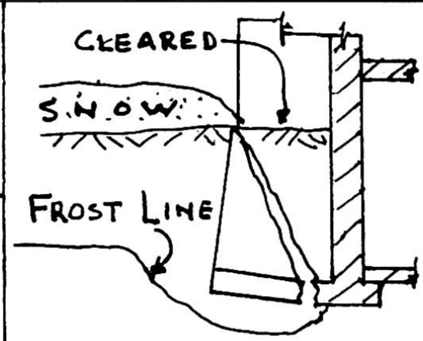
ICE FORCES ACTING ON RETAINING WALL



SPALLING/DISINTEGRATION DUE TO PENETRANT WATER FREEZING



MELTING OF PERMA-FROST BY HEAT CONDUCTION



DAMAGE DUE TO UNNATURAL FROST DEPTH

ILLUSTRATIONS OF DIRECT AND INDIRECT EFFECTS OF SNOW AND ICE ON STRUCTURES

Seminar IX

Entwicklungen bei der Planung von Stahlbeton- und Spannbetonbauwerken

Developments in the Design of Reinforced and Prestressed Concrete Structures

Développements dans le projet de constructions en béton armé et précontraint

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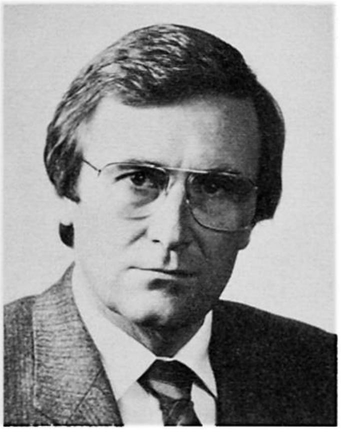


Manfred Wicke, geboren 1933, promovierte als Bauingenieur an der TU Wien. Er arbeitete 12 Jahre in einer grossen Bauunternehmung, deren Konstruktionsbüro er ab 1963 leitete. Seine Tätigkeit umfasste den konstruktiven Ingenieurbau, insbesondere Brückenbau. Seit 1971 ist er ordentlicher Universitätsprofessor für Stahlbeton- und Massivbau an der Universität Innsbruck und als Prüfingenieur tätig.

ZUSAMMENFASSUNG
Die Zuverlässigkeitstheorie und die Plastizitätstheorie sind gedanklich ausgereift, und stehen vor einer breiten Anwendung in der Konstruktionspraxis. Bisherige Erfahrungen mit der Handhabung der neuen Theorien und dem Verhalten der danach errichteten Bauwerke sind somit von Interesse. Die Beobachtung von Bauwerken, insbesondere Spannbetonbrücken, zeigt unter den tatsächlichen Gebrauchslasten mitunter ein Verhalten, das von den Erwartungen abweicht. Dies sollte zum Überdenken des bisherigen Grundgedanken des Spannbetons Anlass geben. Bei der Bewehrungsführung ist zu erkennen, dass unsere bisherigen, auf bestimmte Fragestellungen ausgerichteten Merkregeln, auf eine einheitliche gedankliche Grundlage abgestellt werden.

SUMMARY
The reliability theory and the plasticity theory are now known well enough to be applied widely in the daily work of the engineer. Past experiences in the application of new theories and in the behaviour of structures realized according to these theories are of great interest. The observation of structures, in particular prestressed concrete bridges, shows that the real behaviour is different from the expected under the true service loads. This should lead to a reconsideration of the basic ideas in prestressed concrete. It is well known that in the design of reinforcement recommendations used today are based more on a unified basic thought and less on empirically derived approaches.

RESUME
La théorie de la fiabilité et la théorie de la plasticité sont suffisamment développées aujourd'hui et vont être appliquées largement dans la pratique. Les expériences faites avec l'application de nouvelles théories et avec le comportement de structures ainsi calculées sont donc intéressantes. L'observation de constructions, en particulier des ponts en béton précontraint, montrent que sous les charges réelles d'utilisation se produit un comportement différent de celui qui était prévu. Ceci devrait donner lieu à des réflexions sur les idées fondamentales reconnues à ce jour sur le béton précontraint. Il y a lieu de reconnaître, que le choix des armatures dites constructives quitte le domaine essentiellement empirique pour se baser de plus en plus sur des considérations rationnelles uniformes.

1. EINLEITUNG

1.1 Der Planungsablauf

Jede Planung ist ein in der Zeit ablaufender Vorgang. Sie beginnt mit den ersten Zielvorstellungen des Bauherrn und reicht bis weit in die Zeit der Ausführung des Bauwerkes hinein. Der Planungsablauf kann als ständig zunehmende Verdichtung der Information verstanden werden. Schlußendlich muß die Information in Beschreibungen, Berechnungen und Plänen soweit aufbereitet sein, daß danach das Bauwerk errichtet werden kann.

Am Planungsprozeß sind unterschiedliche Personen oder Personengruppen beteiligt, die verschiedene Wissensgebiete zu vertreten haben. Grundsätzlich kann man feststellen, daß die Anzahl der Beteiligten wächst, je höher der Anteil der Ausbaukosten an den Gesamtbaukosten wird. Zwischen den Beteiligten muß der Informationsfluß sichergestellt sein, wobei diese Aufgabe in der Regel von einem der Planenden treuhändig für den Bauherrn übernommen wird.

Die Planung hat das vom Bauherrn gesetzte Ziel zu verfolgen, und dabei die vorhandenen Randbedingungen zu beachten. Dazu zählen die gesetzlichen Bestimmungen und Vorschriften, die jedenfalls einzuhalten sind. Weiters können für bestimmte Bauwerke besondere Auflagen im Interesse der Sicherheit gemacht werden.

Der Planungsablauf gliedert sich in einzelne Phasen, die man beispielsweise als Vorentwurf, Entwurf und Ausführungsprojekt bezeichnen kann. Die im Projekt enthaltene Information nimmt mit fortschreitender Bearbeitung zu. Im Ausführungsprojekt müssen schließlich die Anliegen der unterschiedlichen Planer aufeinander abgestimmt sein.

Im Hochbau übernimmt in der Regel der Architekt oder ein Projektmanagement die Koordination der Planung. Der Ingenieur, der für die Planung der tragenden Struktur verantwortlich ist, tauscht deshalb vornehmlich die Informationen mit dem Architekten aus. Auch die anderen Planer, etwa auf den Gebieten der sanitären Installationen, Heizung, Lüftung, Bauphysik, Elektroanlagen usw. arbeiten auf ihren Fachgebieten eng mit dem Architekten zusammen. Für die Beantwortung der Detailfragen ist der gelegentliche direkte Kontakt zwischen den Betroffenen förderlich.

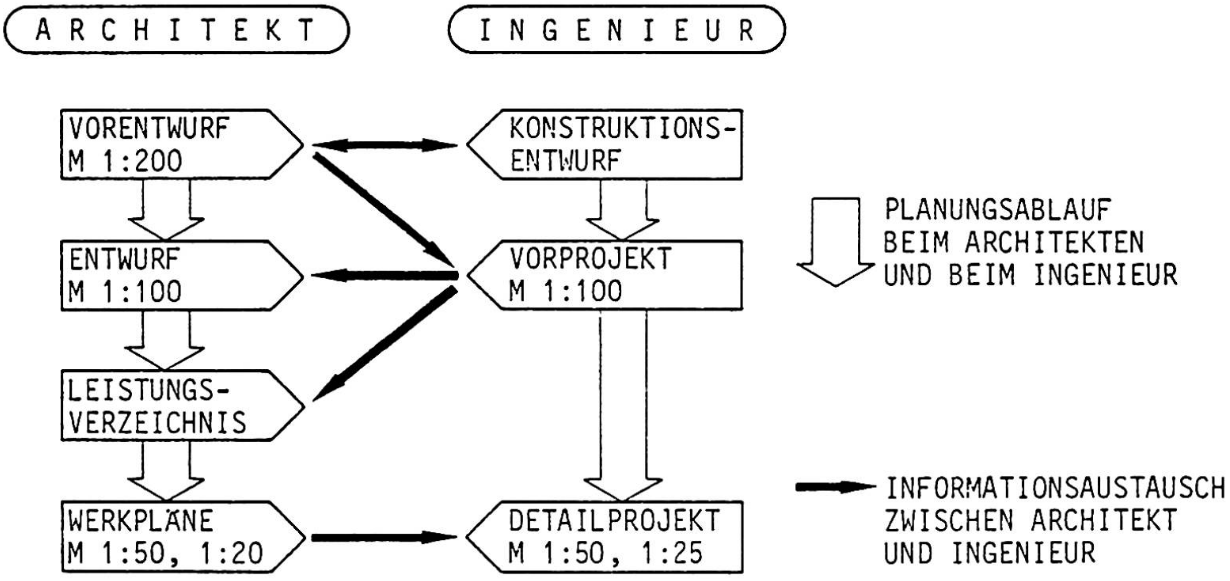


Fig. 1: Planungsablauf im Hochbau

Das in Fig. 1 dargestellte Ablaufschema soll beispielshaft den Informationsaustausch zwischen dem Architekten, als Koordinator, und dem Ingenieur, als dem Planer der Konstruktion darstellen. Der zunehmende Gehalt an Informationen spiegelt sich in der fortschreitenden Vergrößerung des Maßstabes wider. Eine Vielzahl anderer Abläufe ist möglich. Von Land zu Land finden sich Unterschiede, die meist in den besonderen wirtschaftlichen, politischen oder rechtlichen Umständen begründet sind.

Im Brückenbau, bei dem die tragende Konstruktion zum maßgebenden Einfluß wird, sind die Verhältnisse einfacher. Meist verfügt der Bauherr, in der Regel eine Straßenverwaltung, über langjährige Erfahrungen und hat folglich sehr bestimmte Vorstellungen über die Brückenausrüstung, wie Abdichtung, Fahrbahnbelag, Geländer, Entwässerung usw. Abgesehen von diesen Vorgaben ist der planende Ingenieur jedoch frei in der Wahl des Tragwerkes und der Baustoffe. Er kann sich ganz von den Gesetzen der Baustatik und der Festigkeitslehre, den Regeln der Ästhetik und den Erfordernissen der Bauausführung und der Erhaltung leiten lassen. Es ist für den störungsfreien Planungsablauf wichtig, die Vereinbarungen über die Zuständigkeiten und den Informationsfluß einzuhalten. Falls keine allgemein gültigen Regelungen darüber bestehen, sind für ein Bauvorhaben besondere Festlegungen zu treffen. Dies ist letztlich im Hinblick auf die Sicherheit, zur Vermeidung von Fehlern und Irrtümern, nötig.

1.2 Planung von Stahlbeton- und Spannbetontragwerken

Im Allgemeinen beginnt die Planung der Konstruktion, und damit die Planung von Stahlbeton- und Spannbetontragwerken, erst zu einem verhältnismäßig späten Zeitpunkt der Gesamtplanung. Die bis dahin erfolgten Entscheidungen stellen für den Konstruktionsentwurf meist unabänderliche Anfangsbedingungen dar. Es ist deshalb wünschenswert, mit der konstruktiven Planung möglichst frühzeitig zu beginnen, um in die grundlegenden Entscheidungen die Erfordernisse der Konstruktion bereits einfließen lassen zu können. Im Hochbau ist eine erste Kontaktaufnahme zwischen Architekt und Ingenieur bereits beim Vorentwurf anzustreben, und zwar umso eher, je anspruchsvoller das Projekt im konstruktiven Bereich ist (siehe Fig. 1). Wenn dies nicht der Fall ist, beschränkt sich die Tätigkeit des Ingenieurs auf das Ausrechnen und Durchzeichnen von Tragwerken, die Fachfremde festgelegt haben. Dies ist weder sachdienlich, noch im Interesse des Bauherrn gelegen, der zu recht bestrebt sein muß, das Gesamtwissen des Ingenieurs für sein Projekt zu nutzen.

Neben den Anfangsbedingungen, die sich aus der Funktion des Bauwerkes ergeben, sind die durch Gesetze und Vorschriften gestellten Randbedingungen einzuhalten. Leider engen sehr weitreichende Vorschreibungen in Bauordnungen und Normen den Freiheitsraum des Ingenieurs vielfach weitgehend ein.

Im Zuge der Planung von Stahlbeton- und Spannbetontragwerken sind sehr verschiedenartige Entscheidungen zu treffen. Die wichtigsten Fragen lassen sich nachstehenden Gruppen zuordnen.

- Einwirkungen aus der geplanten Nutzung
- unvermeidbare Umwelteinflüsse
- außergewöhnliche Einwirkungen
- Auswahl der Bauweise (Stahlbeton : Spannbeton)
- Auswahl der Baustoffe
- Auswahl der Art des Tragwerkes
- Einflüsse des Bauverfahrens
- Erfordernisse der Wartung und Erhaltung
- statische Modellbildung und Berechnung
- konstruktive Durchbildung
- Bewehrungsführung

Während die ersten, grundlegenden Entscheidungen sehr frühzeitig zu treffen sind, reicht es aus, die letztgenannten erst während der Detailplanung zu finden.

2. EINWIRKUNGEN

Zu den Einwirkungen zählen Lasten und Zwänge. Für die Auswahl der Art des Tragwerkes ist es bedeutend, ob Lasten oder Zwänge überwiegen. Bei der ausschließlichen Einwirkung von Lasten reicht es aus, dem Tragwerk entsprechende Festigkeit zu verleihen. Bei Zwängen sind Festigkeit und Verformbarkeit aufeinander abzustimmen.

2.1 Einwirkungen aus der geplanten Nutzung

Dabei handelt es sich überwiegend um Lasten, und zwar um ständige und veränderliche. Die ständigen Lasten aus dem Eigengewicht des Tragwerkes und aus den Ausbaulasten sind gut erfaßbar. Abweichungen werden im wesentlichen durch die Ausführungstoleranzen bestimmt. Bei den veränderlichen Lasten hat man sich in der Vergangenheit mit der Erfassung von Höchstwerten begnügt. Neue Impulse gingen von der probabilistischen Sicherheitstheorie aus. Aus der geforderten Zuverläßlichkeit eines Bauwerkes lassen sich Anforderungen an die oberen Fraktilwerte des Belastungskollektives festlegen. Mit Hilfe der statistischen Auswertung von erhobenen Belastungen können die charakteristischen Werte der Belastung bestimmt werden. Für einige häufige Gebäudefunktionen, wie beispielsweise Wohnbauten, sind derartige Auswertungen bereits erfolgt. Für viele andere Funktionen stehen sie noch aus, weshalb man weiterhin auf normativ festgelegte, nominelle Belastungen angewiesen ist. Umfangreiche Arbeiten in der Erfassung und statistischen Auswertung von Belastungen müssen noch erfolgen.

Verfügt man über eine entsprechende Datenmenge, dann kann man diese nicht nur hinsichtlich der oberen Fraktilwerte,sondern auch hinsichtlich dauernd wirkender und häufig auftretender Lastanteile auswerten. Letztere haben zwar keinen Einfluß auf die Tragsicherheit, sehr wohl aber auf die Gebrauchsfähigkeit und die Ermüdung.

2.2 Einwirkungen aus Umwelteinflüssen

Zu diesen unvermeidbaren Einwirkungen zählen die Belastungen durch Schnee und Wind, die Anfachung von Schwingungen durch Erdbeben oder künstliche Erschütterungen und der Einfluß von Temperatur. In jenen Ländern, in denen seit Jahrzehnten meteorologische und seismische Aufzeichnungen geführt werden, stehen die benötigten Daten in ausreichendem Maße zur Verfügung. Dies gilt auch für die täglichen und jahreszeitlichen Schwankungen der Lufttemperatur, jedoch nicht im gleichen Maße für die zeitlich veränderlichen Temperaturfelder in Bauwerken. Diese sind zwar grundsätzlich einer Berechnung zugänglich, doch wären systematische Versuche, an denen die Rechenparameter geeicht werden könnten, sehr zweckdienlich. Zur stärksten Erwärmung führt die Sonneneinstrahlung, doch sind gerade dafür wesentliche Einflüsse, wie etwa die Beschaffenheit der Betonoberfläche ungeklärt.

2.3 Außergewöhnliche Einwirkungen

Von der Natur solcher Ereignisse her sind diese nur für die Tragsicherheit von Interesse. Neben der grundsätzlichen Schwierigkeit derartige Einwirkungen zahlenmäßig festzulegen ist die Frage von Bedeutung, mit welchen Einwirkungen gemäß 2.1 und 2.2 die außergewöhnlichen Einwirkungen zu überlagern sind.

3. TRAGFÄHIGKEIT

Zum Nachweis ausreichender Tragsicherheit wird die probabilistische Sicherheitstheorie immer häufiger herangezogen. Die Umstellung vieler nationaler Normenwerke auf diese neue Grundlage ist im Gange oder bereits abgeschlossen. Auch die Plastizitätstheorie hat nach mehr als zwei Jahrzehnten wissenschaftlicher Überlegungen einen Stand erreicht, der eine Anwendung in der Praxis zuläßt. Eine direkte Überprüfung beider Theorien an ausgeführten Bauwerken ist naturgemäß nicht möglich. Dennoch wären Erfahrungen über das Verhalten von Bauwerken, die nach diesen Theorien errichtet wurden, sehr erwünscht. Besonders wertvoll wäre das Verhalten über längere Zeiträume.

Im 19. Jahrhundert wurde unser bisheriges deterministisches Sicherheitssystem ausgeformt und hat die bis dahin verwendete empirische Bemessung abgelöst. Dieser Übergang vollzog sich, obwohl das alte Verfahren zur Bemessung der bewährten Bauweisen, wie Mauerwerk und Holzbalkendecken, auch weiterhin geeignet gewesen wäre. Es waren keineswegs Rückschläge, sondern die neu auftretenden Bauweisen, der Stahlbau und der Stahlbetonbau, die die Einführung der rechnerischen Bemessung erforderlich machten. Für diese Bauweisen gab es keine überlieferten Bemessungsregeln, die auf langjährige Erfahrung abgestellt waren. Dies führte dazu, daß neue Wege beschritten werden mußten.

Aus Versuchen und deren gedanklicher Deutung wurden Bemessungsverfahren entwickelt, die eine Voraussage des Tragvermögens gestatteten. Dieses Verfahren konnte auf verschiedenartige Bauformen und Bauteile angewandt werden, während das alte Verfahren wenig Spielraum für die Abwandlung überlieferter Bauformen zuließ. Offensichtlich war dies einer von mehreren Gründen für den raschen Siegeszug der Stahlbetonbauweise. Die Rechtfertigung der rechnerischen Bemessung kam allerdings erst im Laufe der Zeit: Nachdem aufgrund langjähriger Erfahrungen feststand, daß die Zuverlässigkeit der nach diesem Verfahren bemessenen Bauwerke gegeben war, konnte man mit gutem Gewissen sagen,daß sich der Übergang bewährt hatte.

Die heutige Situation ist der damaligen ähnlich. Der Wechsel von der deterministischen zur probabilistischen Sicherheitstheorie wird keineswegs durch etwaige Rückschläge im bisherigen Erfahrungsbereich notwendig. Hier hätte die bisherige empirische Festlegung der Sicherheitsfaktoren und deren Überprüfung an der in der Praxis auftretenden Schadensrate auch weiterhin genügt. Es waren vielmehr völlig neue Bauaufgaben, wie beispielsweise Kernkraftwerke und Off- Shore-Konstruktionen, für die keine Erfahrungen über das nötige Sicherheitsniveau vorlagen. Daraus entstand die Notwendigkeit,den Begriff der Sicherheit rational zu fassen und zu quantifizieren. Mit der Zuverlässigkeitstheorie gelingt es,Vorhersagen über die Sicherheit von Bauaufgaben zu machen, die außerhalb des bisherigen Erfahrungsbereiches liegen. Es ist naheliegend, die dafür gefundene Theorie auch auf den bisherigen Erfahrungsbereich anzuwenden und damit die empirisch begründeten Sicherheitskoeffizienten auf eine rationale Grundlage zu stellen. Aufgrund der sorgfältigen Vorbereitung der Umstellung des Sicherheitssystems darf man mit dessen Erfolg rechnen.

In der neuen Sicherheitstheorie können die bisherigen, sehr gründlichen Kenntnisse über die Baustoffe Stahl und Beton weiter verwendet werden. Lediglich die Rechenwerte für die Festigkeit müssen auf die Erfordernisse der Wahrscheinlichkeitsbetrachtung abgestimmt werden. Für die Berechnung des Querschnittswiderstandes stehen somit sehr ausgereifte Theorien zur Verfügung.

Durch neuere Entwicklungen im Tunnelbau gewann die Frage nach der Tragfähigkeit sehr junger Betonkonstruktionen an Bedeutung. Der erwünschte Tagestakt bei der Erstellung der Ringe der Innenschale erfordert Ausschalfristen von 12

Stunden und darunter. Zu diesem frühen Zeitpunkt sind die Festigkeit und der Verformungswiderstand in rascher Entwicklung. Der Anteil der bleibenden Verformungen ist sehr hoch. Eine weitere Klärung der vielschichtigen Zusammenhänge zwischen der Abbindetemperatur und den Eigenschaften des jungen Betons wäre sehr willkommen.

4. NACHWEISE UNTER GEBRAUCHSLAST

Die Gebrauchsfähigkeit eines Tragwerkes stellt für seine Besitzer und für seine Benützer eine wesentliche Voraussetzung für die ordnungsgemäße Nutzung dar. Die große Anzahl von Veröffentlichungen über die Tragsicherheit, die in den vergangenen Jahren erschienen ist, darf nicht zu der Annahme verleiten, daß die Tragfähigkeit die ausschließlich maßgebende Eigenschaft eines Bauwerkes wäre. Die eingehende Behandlung der Zuverläßigkeitstheorie und der Plastizitätstheorie in der Fachwelt ist offensichtlich auf die Tatsache zurückzuführen, daß die im vorangegangenen Kapitel beschriebene Umstellung des Sicherheitssystems vorbereitet wurde.

Die Zuverläßigkeit eines Tragwerkes wird durch die Einhaltung sehr geringer Versagenswahrscheinlichkeiten überprüft. Je nach der gestellten Aufgabe sind Werte zwischen 10^{-5} und 10^{-7} , und auch darunter, einzuhalten. Das Versagen eines Tragwerkes wird damit zu dem angestrebten, äußerst seltenen Ereignis von dem der einzelne Bürger praktisch nie betroffen ist. Mangels eigener Erfahrungen mit der Tragsicherheit wird für den Laien dieser Begriff nicht zu einem Kriterium, nach dem ein Bauwerk beurteilt wird, sondern sie bleibt Bestandteil seines allgemeinen Sicherheitsbedürfnisses. Die Menschen machen ihre Erfahrungen mit den Bauwerken aufgrund ihrer tagtäglichen Beobachtungen. Diese betreffen aber nicht ein äußerst seltenes, sondern das sehr wahrscheinliche Verhalten eines Tragwerkes, und zwar unter den tatsächlich vorhandenen Gebrauchslasten. Der Erfolg oder Mißerfolg eines Bauwerkes wird von der Öffentlichkeit in starkem Maße nach dessen Verhalten unter Gebrauchslast beurteilt. Wir Ingenieure sind gut beraten, wenn wir deshalb der Gebrauchsfähigkeit bei der Planung den entsprechenden Stellenwert einräumen.

Die Beanspruchungen unter Gebrauchslast waren, vor Einführung des Grenzzustandes der Tragfähigkeit, jahrzehntelang das maßgebende Bemessungskriterium. Man möchte deshalb meinen, daß die Nachweise unter Gebrauchslast abgeklärt und unproblematisch seien. Dennoch lehrt uns die Langzeiterfahrung, insbesondere an Spannbetonbrücken, daß dies nicht so ist. Der ursprüngliche Grundgedanke des Spannbetons, alle Fasern des Betonquerschnittes auf Dauer unter Druckspannungen zu setzen, wurde bald durch die Duldung von Zugspannungen, die mit ausreichender Sicherheit unter der Betonzugfestigkeit lagen, erweitert. Nach diesem Grundgedanken sollte das Tragwerk rissefrei bleiben und folgerichtig wurden die Gebrauchsspannungen mit den Werten des ungerissenen Querschnittes berechnet.

Die eingehende Beobachtung von Spannbetonbrücken in der jüngsten Vergangenheit hat jedoch gezeigt, daß Risse vorhanden sind, und zwar sowohl in voll vorgespannten, als auch in beschränkt vorgespannten Tragwerken. Es erscheint somit wenig sinnvoll diese beiden Vorspannklassen künftig beizubehalten, es sei denn, man möchte zwischen geringerer und höherer Rißwahrscheinlichkeit unterscheiden.

Die eben genannten Untersuchungen wurden von den sogenannten Koppelfugenschäden ausgelöst. Darunter versteht man das Öffnen von Koppelfugen zwischen zwei Bauabschnitten, das auch als ein Riß am Ort einer abgeminderten Zugfestigkeit gedeutet werden kann. Die Untersuchungen zeigten dann eine Häufung von Rissen im Bereich der Momentennullpunkte von Durchlaufträgern und Rahmenriegeln. Die Analyse der Tragwerke ergab, daß häufig Zwänge die Ursache der Risse waren.

Solche Zwänge sind Auflagersetzungen oder Temperaturgradienten. Sie waren ursprünglich gar nicht oder in unzureichender Größe in Rechnung gestellt worden. An den Koppelfugen treten noch zwei weitere Ursachen hinzu. Zunächst die örtlichen Verformungen und die daraus entstehenden Scheibenspannungen aus dem Anspannen und späteren Entlasten der Koppelanker. Weiters die erhöhten Kriechverluste an Vorspannkraft zufolge erhöhter Betonspannungen, die sich aus dem durch die größeren Hüllrohre geschwächten Betonquerschnitt ergeben.

Eine wirklichkeitsnahe Beschreibung des Gebrauchszustandes muß alle tatsächlichen Einwirkungen richtig erfassen. Dies gilt in besonderem Maße für die Zwänge, weil sich die Schnittkraftverteilungen unter Zwang von jenen unter Last grundsätzlich unterscheiden. Zur Verdeutlichung ist in Fig. 2 der Verlauf der Biegemomente

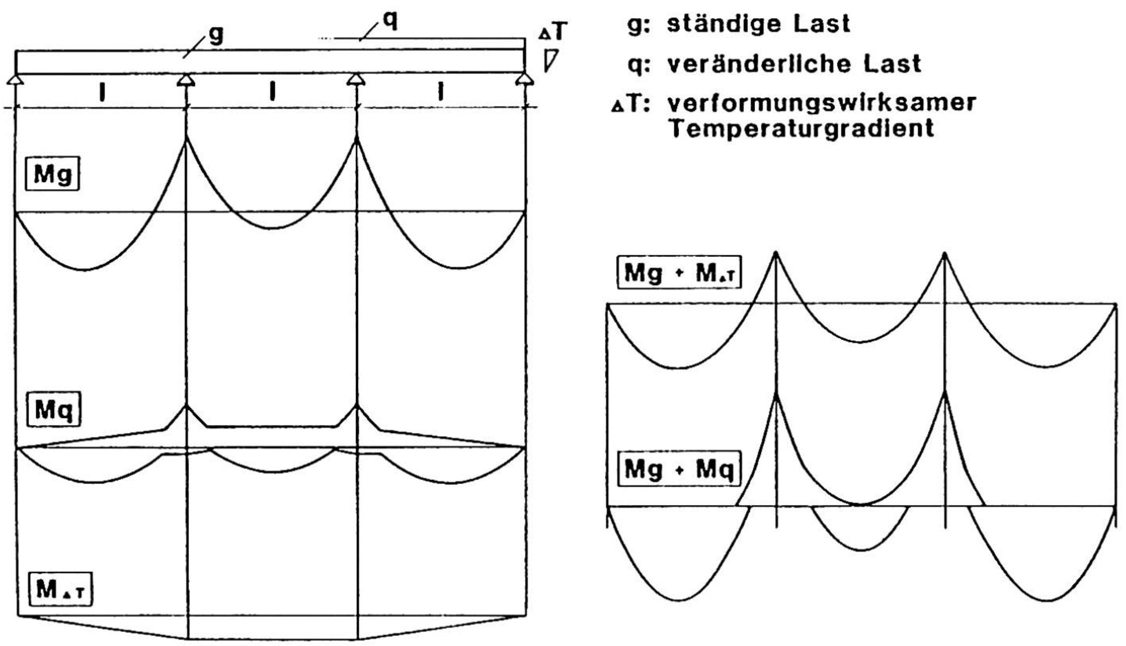


Fig. 2: Momentenverlauf unter Last (g, q) und Zwang (ΔT)

in einem Dreifeldträger unter ständiger Last g, unter veränderlicher Last q und unter einem positiven Temperaturgradienten ΔT beispielshaft dargestellt. Man erkennt, daß im Bereich der Momentennullpunkte mit den größten prozentuellen Fehlern zu rechnen ist, wenn die Zwangswirkung vernachlässigt wird. Man sieht aber auch deutlich, warum in Feldmitte die Rißwahrscheinlichkeit geringer ist, obwohl die Zwangsmomente im ganzen Mittelfeld gleich groß sind: Der Feldquerschnitt ist für die größten Nutzlastmomente bemessen, deren gleichzeitiges Auftreten mit den Temperaturzwängen aber unwahrscheinlich ist. Unter wahrscheinlichen Nutzlasten bleiben noch Reserven zur Aufnahme von Zwängen übrig. Ähnliche Reserven sind im Bereich der Momentennullpunkte nicht vorhanden.

Um die Anrißgefahr richtig abschätzen zu können, muß man wahrscheinliche Einwirkungen kombinieren. Dazu sind zunächst Angaben über häufige und größte Temperaturgradienten unter den jeweiligen klimatischen Bedingungen nötig. Für die Berechnung des Zwangsmomentes ist der lineare, verformungswirksame Anteil ΔT gemäß Fig. 3 heranzuziehen.

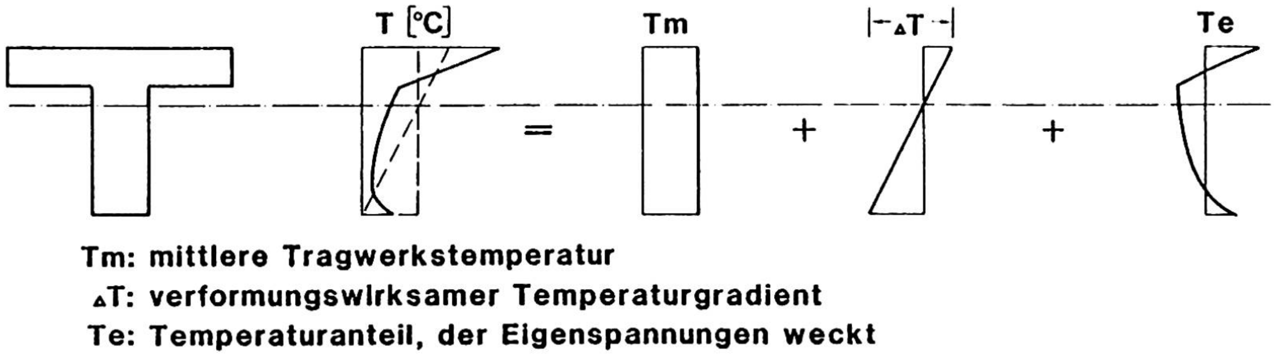


Fig. 3: Aufgliederung des Temperaturverlaufes

Beiträge über Messungen an Bauwerken und deren Auswertung, sowie über die Überlagerung der Lastfälle zu wahrscheinlichen Kombinationen sind von Interesse. Auch die Frage der Biegesteifigkeit des Tragwerkes sollte angeschnitten werden. Wie in Kapitel 7 "Spannbeton" noch näher ausgeführt wird, erscheint es gar nicht zweckmäßig, die Temperaturspannungen zu überdrücken, um die Anrißgefahr auszuschließen. Wenn man aber auch im Spannbeton Risse zuläßt, dann gewinnt der Ansatz der richtigen Biegesteifigkeit, insbesondere zur Berechnung der Zwangsmomente, an Bedeutung.

5. DAUERHAFTIGKEIT

Der Planende kann durch Maßnahmen auf unterschiedlichen Gebieten günstig auf die Dauerhaftigkeit einwirken. Solche Gebiete sind für normale Umweltbedingungen:

- die Auswahl des Tragwerkes
- die Betontechnologie
- die Betondeckung

Unter aggressiven Umweltbedingungen, oder künstlichen chemischen Angriffen kommen hinzu:

- der Schutz der Betonoberfläche durch Anstriche, Beschichtungen, Abdichtungen und dergleichen
- korrosionsgeschützte Bewehrungen

Zur Zeit herrscht auf dem Gebiet der Dauerhaftigkeit eine rege Tätigkeit. Nationale und internationale Organisationen befassen sich damit, wobei Fragen der physikalischen und chemischen Angriffe und die Widerstandsfähigkeit des Betons gegen diese Angriffe im Vordergrund stehen. Im Sinne der Zielsetzungen der IVBH im allgemeinen und des Themas der Sitzung IX im besonderen sollten die Beiträge die Möglichkeiten des Planenden, die Dauerhaftigkeit günstig zu beeinflussen, bevorzugt behandeln.

Aus der Sicht der Dauerhaftigkeit ist darauf zu achten, daß die Tragwerke einfach gewartet und ausgebessert werden können. Weiters ist zu erwähnen, daß steife, verformungsunwillige Systeme und Querschnitte eher zu klaffenden Rissen neigen als weiche Tragwerke.

Die Betontechnologie und die Betondeckung müssen aufeinander abgestimmt sein, um den Korrosionsschutz der Bewehrung zu sichern. Zur weiteren Klärung dieser Fragen sind Beobachtungen and Bauwerken, Versuchsergebnisse und Rechenmodelle über das Eindringen der Karbonatisierungsfront in den Beton von Bedeutung. Auf dieser Basis könnten auch Konzepte zur Abschätzung der Lebensdauer eines Tragwerkes erarbeitet werden. Kennt man jedoch den Einfluß bestimmter Vorkehrungen - wie beispielsweise der Stärke der Betondeckung oder der Größe des Zementgehaltes des Betons - auf die Lebensdauer, dann kann man die Wirtschaftlichkeit dieser Maßnahmen überprüfen, indem man die zusätzlichen Kosten auf die verlängerte Lebensdauer bezieht.

Für den aktiven Schutz des Betons und der Bewehrungen vor chemischen Angriffen sind Langzeiterfahrungen mit bestimmten Schutzmaßnahmen von Bedeutung. Für bestimmte Bauteile, wie beispielsweise Gehwegkappen von Straßenbrücken, könnte die Verwendung von epoxybeschichteten Bewehrungsstäben durchaus zielführend sein.

6. KONSTRUKTIVE DURCHBILDUNG

In den Konstruktionsregeln nationaler und internationaler Vorschriften, Richtlinien und Merkblätter ist die jahrzehntelange Erfahrung mit einer bestimmten Gestaltung der Bauteile und der Bewehrungsführung niedergelegt. In der Vergangenheit wiesen die Konstruktionsregeln der einzelnen Länder mitunter beträchtliche Unterschiede auf, die durch unterschiedliche Überlieferungen begründet waren. Regelungen, die auf langjähriger Erfahrung beruhen, sind jedoch starr und schwer veränderbar. Jede Abweichung bedeutet ein Verlassen des bisherigen Erfahrungsbereiches. Der Erfolg oder Mißerfolg einer Neuerung kann oft erst nach jahrelanger Erprobung beurteilt werden. Zu einer Angleichung der nationalen Traditionen kam es in den Sechziger- und Siebzigerjahren durch den internationalen Erfahrungsaustausch im Rahmen von CEB und FIP. Die derart vereinheitlichten Konstruktionsregeln sind jedoch weiterhin vorwiegend in der Erfahrung begründet, und somit weiterhin schwer veränderbar.

Diese Lage legt den Gedanken nahe, das Konstruieren durch gedankliche Modelle zu begründen. Die Aussagen solcher Modelle können zunächst am vorhandenen Erfahrungsschatz überprüft werden. In der Folge können mit diesen überprüften Modellen Vorhersagen über Neuerungen gemacht werden. Damit erhielte die Entwicklung auf dem Gebiet der Konstruktionsregeln neue Anregungen und könnte überdies beschleunigt werden. Ein weiterer Vorteil bestünde darin, daß die Regeln gedanklich nachvollziehbar und damit erlernbar würden. Unsere bisherigen Merkregeln könnten aus unserem Wissensstand deduktiv abgeleitet werden.

Die eben dargelegten Gedanken sind nicht neu und werden für die Lösung bestimmter Detailaufgaben bereits mit Erfolg angewandt. Als Beispiel diene die Beanspruchung eines Balkensteges durch Biegung und Querkraft, wie sie in Fig. 4 dargestellt ist.

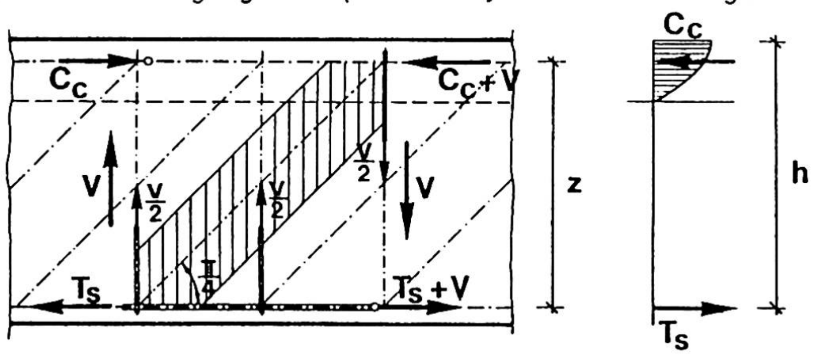


Fig. 4: Druckfeld in einem Balkensteg

Bereits das klassische Fachwerksmodell, das später zum Druckfeld weiterentwickelt wurde, ließ erkennen, daß die Bügel über die ganze Steghöhe gleich beansprucht waren, und ihre Zugkraft in die Knoten an den Gurten eintragen mußten. Damit wurde verdeutlicht, daß eine gute Endverankerung der vertikalen Bügelschenkel konstruktiv notwendig ist.

In [1] wird der Versuch unternommen, die für verschiedene Detailaufgaben vorhandenen Modelle zu sammeln, zu ordnen und auf eine gemeinsame gedankliche Basis zu stellen. Der einheitliche Lösungsweg sei anhand von Fig. 5 für das Spaltzugproblem beschrieben.

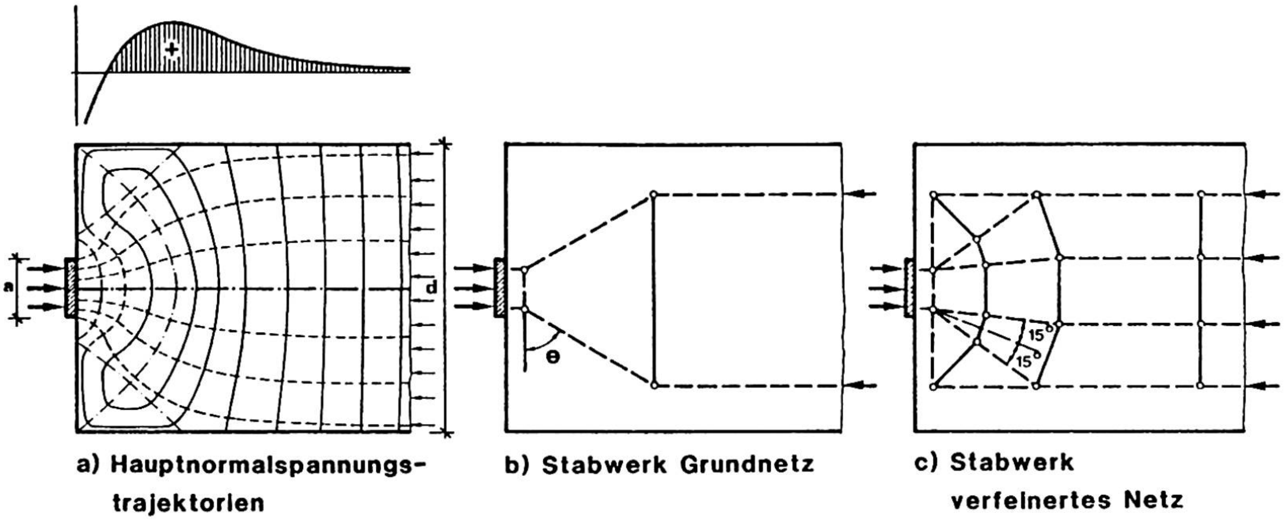


Fig. 5: Modellbildung

Ausgangspunkt ist jeweils das Feld des Spannungstensors, das aufgrund der Elastizitätstheorie ermittelt und durch seine Hauptnormalspannungstrajektorien und die Spannungsverteilung in maßgebenden Schnitten anschaulich dargestellt wird. Dem Trajektorienbild wird ein Stabwerk angepaßt, dessen Stabkräfte aufgrund einfacher Gleichgewichtsüberlegungen bestimmt werden. Die Zugkräfte werden in der Regel durch Bewehrung abgedeckt. Die Druckstreben sind jedoch keine prismatischen Stäbe, ihre Gestalt wird gleichfalls durch die natürliche Lastausbreitung im Beton bestimmt. Die Bemessung der Druckstäbe erfolgt aufgrund der Spannungen des ebenen Spannungszustandes mit Hilfe der Bruchhypothese von MOHR. Den Einfluß der Kraftteinschnürung d/a auf die Tragfähigkeit der Druckstrebe zeigt Fig. 6.

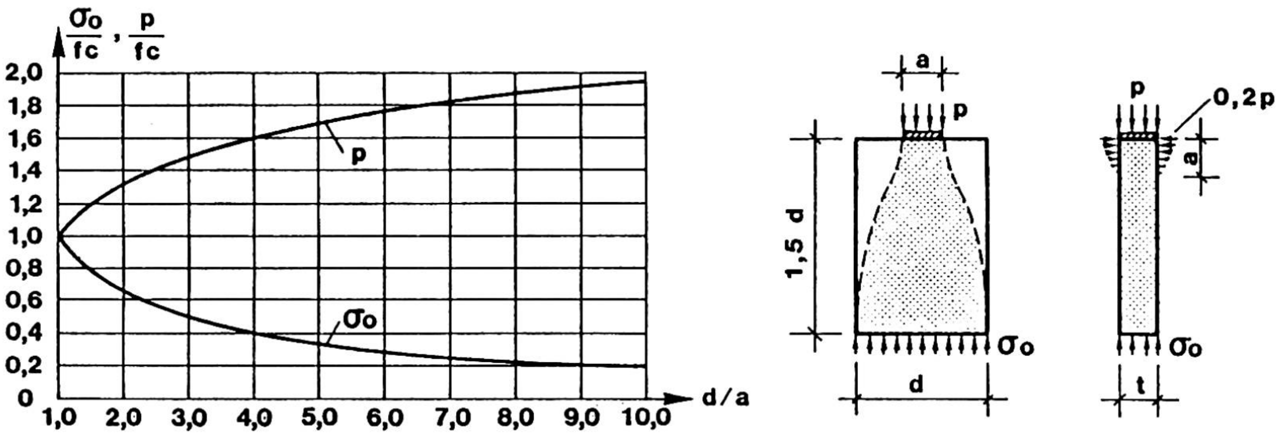
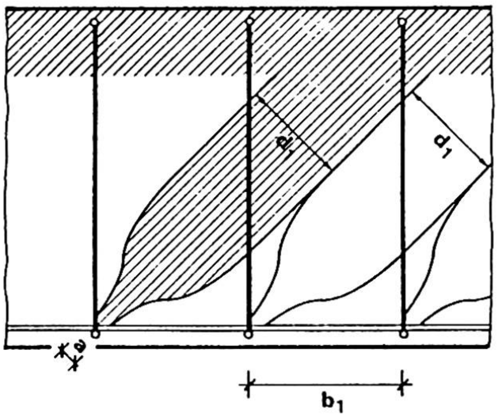


Fig. 6: Tragfähigkeit der Druckstrebe

Das beschriebene Verfahren geht von der Elastizitätstheorie aus, und läßt Abweichungen von den Hauptrichtungen nur in begrenztem Maße zu. Die dafür benötigte geringe Rotationsfähigkeit ist stets vorhanden.

Das Modell erfüllt immer die Gleichgewichtsbedingungen und orientiert sich wegen der begrenzten Umlagerungen an den Verträglichkeitsbedingungen. Damit werden klaffende Risse unter Gebrauchslast vermieden.

In Fig. 7 wird das Modell der Lastausbreitung auf die Druckstrebe des Fachwerkmodelles von Fig. 4 angewandt.



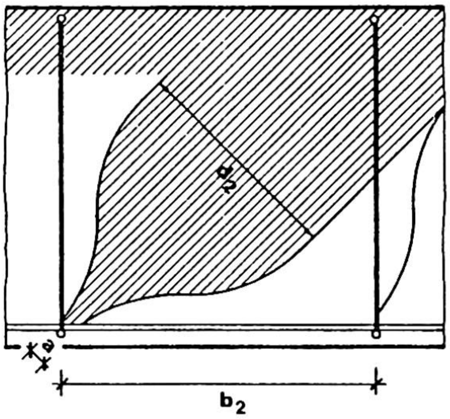


Fig. 7: Einfluß des Bügelabstandes auf die Tragfähigkeit des Druckfeldes

Die Lasteintragungsbreite a ist konstruktiv vorgegeben. Die Breite der Druckstrebe richtet sich nach dem Bügelabstand. Bei sonst gleichen Voraussetzungen ergibt die geringere Lastausbreitung ($d_1/a < d_2/a$) bei engem Bügelabstand eine größere Tragkraft für das Druckfeld.

Mit der kurzen Beschreibung des Verfahrens soll die Anregung für weiterführende Beiträge gegeben werden. Ebenso wären Berichte willkommen, die die gedankliche Begründung auf anderem als dem aufgezeigten Weg behandeln.

Ein gänzlich anders gearteter Fragenkreis ergibt sich aus der wirtschaftlichen Notwendigkeit die Bewehrungsarbeiten weiter zu rationalisieren. Die angestrebte Einsparung an Arbeitszeit kann durch stärkere Verwendung von nur geschnittenen, jedoch nicht gebogenen Bewehrungsstäben erfolgen. Weiters ist die Anzahl der verwendeten Biegeformen zu verringern, damit der Einsatz von Biegeautomaten sinnvoll wird.

7. SPANNBETON

Wie bereits in Kapitel 4 gezeigt wurde, konnte das bisher im klassischen Spannbeton angewandte Nachweisverfahren die angestrebte Rissefreiheit nicht mit Sicherheit erreichen. Als hauptsächliche Ursache dafür wurden nicht in Rechnung gestellte Zwänge erkannt. Nach dieser negativen Rückmeldung aus der Beobachtung ausgeführter Bauwerke ist zu überlegen, ob das ursprüngliche Konzept beibehalten oder durch ein neues ersetzt werden soll.

Ein Bauwerk wird für eine geplante Funktion entworfen, und sein Tragwerk dient vornehmlich dazu,die Lasten aus der geplanten Nutzung und die für diese Nutzung benötigten ständigen Lasten,in den Baugrund abzutragen. Die gleiche Forderung gilt auch für die Wind- und Schneelasten.

Weiters kann ein Tragwerk unvermeidbaren Verformungen, die ihm von der Umgebung aufgeprägt werden ausgesetzt sein.

Diesen Verformungen können nur statisch bestimmte Tragwerke ohne innere Anstrengung folgen. In allen statisch unbestimmten Tragwerken werden dadurch Beanspruchungen geweckt, die in der Regel unerwünscht sind, weil sie einen zusätzlichen Materialaufwand erfordern. Bei einem Balken, der einem Biegezwang unterworfen ist, sind die geweckten Schnittgrößen der Biegesteifigkeit des Balkens verhältnisgleich. Mit der Rissebildung sinkt die Biegesteifigkeit stark ab, womit auch die Zwangsschnittgrößen im gleichen Verhältnis abgemindert werden. Daraus erkennt man, daß es nicht zweckmäßig ist, die Zwangsmomente durch zusätzliche Vorspannkräfte überdrücken zu wollen.

Durch die angestrebte Rissefreiheit und die damit verbundene größere Steifigkeit des Tragwerkes wird ein Großteil der Zwangsschnittgrößen erst geweckt, die in der Folge den zusätzlichen Materialaufwand verursachen. Es ist zweckmäßiger dem Tragwerk durch Rissebildung die Möglichkeit zu geben die Zwangsmomente abzubauen. Die gerissenen Zonen sind gegenüber den ungerissenen abzugrenzen und mit genügend schlaffer Bewehrung zu durchsetzen, damit keine unzulässig großen Rißbreiten entstehen.

Mit diesem neuen Konzept werden die bisherigen Begriffe "volle Vorspannung" und "beschränkte Vorspannung" inhaltsleer. Man findet damit auch einen nahtlosen Übergang vom Spannbeton zum Stahlbeton. An die Stelle der bisherigen Einteilung, die ausschließlich aufgrund der Betonspannungen oder der Rißbreiten erfolgte, sollte ein Konzept treten, das weitere Gesichtspunkte berücksichtigt. Beispielsweise kann es zweckmäßig sein, die Durchbiegung, oder allgemeiner gesehen die Verformungen, als maßgebenden Gesichtspunkt anzusehen. Für eine bestimmte Bauaufgabe kann es sinnvoll sein, die zeitliche Veränderung der Durchbiegung möglichst klein zu halten. Strebt man hingegen die Wasserdichtigkeit einer Wand an, dann wird man die Rißwahrscheinlichkeit möglichst klein halten, und dazu auch lastunabhängige Spannungen durch Vorspannungen überdrücken. Hat man für eine gestellte Bauaufgabe die geeignete Vorspannkraft ermittelt, dann ist für diese anschließend die Tragfähigkeit und die Gebrauchsfähigkeit nachzuweisen. Reicht die Fließkraft der Spannbewehrung zum Nachweis der Tragfähigkeit nicht aus, ist der Fehlbetrag durch Zulage von Betonstahl abzudecken. Falls unter Gebrauchslast Risse auftreten können, muß der Betonstahl auch die Kontrolle der Rißbreite übernehmen.

Mit dem dargestellten Konzept kann man ein Spannbetontragwerk viel besser den jeweiligen Anforderungen anpassen. Es sind deshalb Berichte über Erfahrungen bei der Anwendung dieser Grundsätze erwünscht.

8. SCHLUSSBEMERKUNG

Es wurde der Versuch unternommen, offene Fragen bei der Planung von Stahlbeton- und Spannbetontragwerken aufzuzeigen. Die schlaglichtartige Beleuchtung einiger Fragen erhebt keineswegs Anspruch auf Vollständigkeit. Auch die getroffenen Bewertungen geben lediglich die persönliche Ansicht des Verfassers wider. Es sind deshalb auch Beiträge zu Themen, die im Einführungsbericht nicht direkt angeschnitten wurden, willkommen.

LITERATURVERZEICHNIS

1. SCHLAICH J., WEISCHEDE D., CEB-BI No. 150, "Detailing of Concrete Structures"

Seminar X

Developments in the Design and Construction of Wood Structures

Développements dans la construction en bois: projet, calcul et exécution

Fortschritte im Ingenieurholzbau

Bo EDLUND
Professor, Tekn. Dr.
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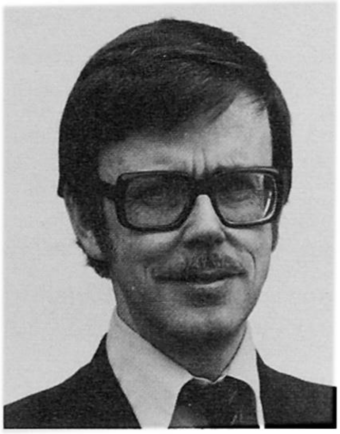
Göteborg, Sweden



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SUMMARY
The field of timber structures comprises questions concerning material properties, fasteners and connections, structural components, structural interaction between components in buildings and bridges, transportation and erection, protection and maintenance. This report focuses the attention on some specific problems which deserve special attention by structural engineers and researchers.

RESUME
Dans le domaine des constructions en bois, il y a des questions concernant les propriétés des matériaux, les liaisons, les éléments structuraux, l'interaction entre différents éléments dans les bâtiments et les ponts, le transport et le montage, la protection des structures et l'entretien. Cet article attire l'attention sur quelques problèmes particuliers méritant l'attention des ingénieurs et des chercheurs.

ZUSAMMENFASSUNG
Im Ingenieurholzbau stellen sich Fragen im Bezug auf Baustoffeigenschaften, Befestigungsmittel und Verbindungen, Bauteile und ihr Zusammenwirken im Hoch- und Brückenbau sowie Probleme aus den Bereichen Transport und Montage, Holzschutz und Unterhalt. Dieser Einführungsbericht macht sowohl den praktisch tätigen Ingenieur als auch den Forscher auf einige Probleme aufmerksam, die spezielle Aufmerksamkeit verdienen.

1. INTRODUCTION

There is a clear trend of increasing the effort to use timber resources more efficiently. This will among others mean a change in methods and material selection with the changing raw material supply. This also forms a background for the need for the development of new or improved methods for the design and construction of wood structures. Since wood species and other conditions are different all over the world the solution might come out differently in various countries.

Developments are expected to take place within the following main areas :

- New or improved knowledge of existing wood materials. Development of new types of wood based materials.
- Better knowledge of the behaviour of fasteners and connections. Development of new types of fasteners and connections.
- Better knowledge of the behaviour of different types of structural elements and structures. Development of new types of structural components and structural systems.
- Progress in standardization of wood structures, design and construction.
- Improved methods for the analysis of timber structures. Increased use of computer aided design (CAD). Improved application of reliability based design.
- Improved methods for wood construction.

At the 11th Congress in Vienna |2| the majority of the contributions to the theme on Timber Structures treated glued laminated timber elements or structures made of such elements. Only one paper in the Final Report concerns fasteners and connections. Therefore, papers on basic problems of timber strength and stiffness related to structures - such as duration of load, moisture effects and fracture mechanics applications - as well as papers on joints and fasteners and on structural components are especially welcome for the coming 12th Congress.

2. MATERIALS

The designer must have basic knowledge of the peculiarities of the natural material wood. Such knowledge is of utmost importance for the understanding of the behaviour of various timber connections and structural components.

2.1 Timber

Better knowledge is still needed of timber as a material both on a microscopic and on a macroscopic scale. Some subjects within this field will, however, fall outside the theme of this paper.
The properties of structural lumber are related to the stress grade obtained by visual or machine stress grading. By in-grade testing the properties of each grade of a certain species are determined for a large number of specimens selected at random.

In many European countries except the United Kingdom hardwood has barely been used for structural purposes. In Germany for example tropical hardwood such as Bongossi has been used for footbridges etc. This material has high strength, but is rather heavy. Some hardwoods have a natural preservation against fungi attack etc.

In some countries experiments with different modifications of wood have been carried out. Some of these modifications aim at structural purposes.

2.2 Wood based materials

There are a few developments concerning the use of new wood based materials, which may serve structural purposes. Typical such products are sheets (plates) of chipboard and flakeboard, fibreboard and plywood.

The use of plywood for structural elements is widespread and well-known. Fibreboard, mainly hardboard, has been increasingly used for many structural applications. Special care is required with respect to moisture influence and creep. Therefore, hardboard-webbed beams are used for rather small structures.

Some interesting possibilities are offered by new types of chipboard and flakeboard, which may be designed with oriented chips, with chips of different sizes and types and layered with respect to strength requirements. The low tensile strength perpendicular to the board plane creates a problem. Long-term behaviour and durability are questions that need further study before such boards can be used with full confidence.

In recent years there has been an increased use of veneer laminates, which are made from veneers of usually 3 to 12 mm thickness glued together with parallel fibre orientation. Although a number of research papers (mainly from the USA) exist on this type of wood-based material more research is needed to determine different properties especially when other species are used.

2.3 Material properties

2.3.1 Stiffness properties

Some effort has been made to establish theoretical models for the determination of the modulus of elasticity from basic material data for a wood-based material such as chipboard. Such models may then be used to simulate the change in stiffness due to changes in different basic parameters.

2.3.2 Theories for wood strength

It has been a tendency in recent years to turn to the questions of basic behaviour of wood at cracking and up to final failure. Some failure stress formulas, which have been popular during a few decades, such as the Norris formula, have been questioned repeatedly. It is therefore important to determine the limitations of such theories and to try to find new ways of describing and predicting timber failure. It has been shown, see e.g. |6| , that the simple empirical formula by Hankinson for typical situations in a highly anisotropic material like wood gives a very good approximation to the actual failure stress.
In 1978 the First International Conference on Wood Fracture was arranged at Banff, Canada |3|. Many of the contributions treat problems concerning wood as an anisotropic, brittle material. It is shown how Fracture Mechanics can be applied to determine wood fracture in different situations. After that Conference further research within this field has been carried out and some of the results ought to be brought forward as contributions to the IABSE Congress, when it now will be held in Vancouver, Canada.

Some paper on analytical studies aiming at either biaxial or even triaxial failure criteria for anisotropic materials is certainly also welcome. Such criteria are often formulated as interaction type formulas including biaxial (or triaxial) stresses or strains and other parameters of importance. They are therefore well suited as a basis for design criteria.

2.4 Load effects

Wood is a viscoelastic material and the long term behaviour is of great importance for the proper design of wood structures. The effects due to duration of load are

of two kinds :
(1) Deflection increase (creep),
(2) Strength reduction.

There have recently been different proposals of viscoelastic fracture mechanics models and cumulative damage theories for the prediction of the influence on strength of duration-of-load combined with different defects such as initial cracks. Papers on such theoretical models should certainly be of interest for the seminar.

2.5 Environmental effects

The environment defines the "working conditions" for timber structures. The air humidity is of special importance both for the strength and stiffness of timber. In recent years, however, there have been several studies, mainly in Canada, showing that for lumber in structural sizes, which has certain defects the influence of high moisture content is much less than for clear wood. Therefore, it should be of great interest to find methods for basic understanding of moisture migration and accumulation in different timber structures used in different situations such as wall and roof structures.

A special problem is created by changing humidity. Often the resulting moisture content variation in the wood is of a periodic nature. Some earlier studies show that certain types of moisture cycling cause a more rapid accumulation of creep deflection than other types. For this problem there is quite a lot of more research to be carried out before we are able to tackle all typical moisture change patterns. The basic mechanism of moisture cycling effects has to be understood first.

3. CONNECTIONS

3.1 Fasteners

Some new types of fasteners such as screws, bolts, nails and staples have been developed in recent years. They are designed to give better and more reliable joints, although many of them are only suitable and economic for certain applications. In some of these cases the new fasteners are only variants of existing types and the actual difference is not great.

A deeper knowledge of the behaviour of different types of fasteners is needed for example for single fasteners, rows of fasteners, for short term and long term loading etc. The influence of moisture content and moisture cycling, especially in combination with creep, are of great interest for the behaviour of mechanical fasteners. A special question concerns fasteners in parallel laminated veneers.

3.2 Metal plate connections and special connections

The metal plate connectors used as nail plates and nailing plates for trusses and similar structures are well known. For special applications different prefabricated special metal plates have been developed for different simple connections such as beam-to-beam connections, see Fig.1, beam-to-column connections and simple moment resistant joints. In some of these connections the plates are nailed through predrilled holes, in others it is a joint with plates and dowels.

For column footings the behaviour of traditional types of cast-in-place rolled steel profiles such as channel section connections has not been investigated properly. New developments include such wood-to-concrete column connections, where glued steel bars are penetrating into the column end and cast into the concrete.

3.3 Glued joints

Some new types of glue are still appearing on the market. Some of them show certain advantages in different applications. The basic knowledge of gluing and glue joint strength, i.e. how to make a reliable and strong glue joint is still unsatisfactory.

The widespread use of finger joints has certainly lead to a better use of timber resources. The knowledge of the behaviour of such joints under long term loading and in varying moisture conditions is still incomplete.

See further the more comprehensive review on timber connections by E.Gehri in section 3 of his report to the 11th Congress |1|.

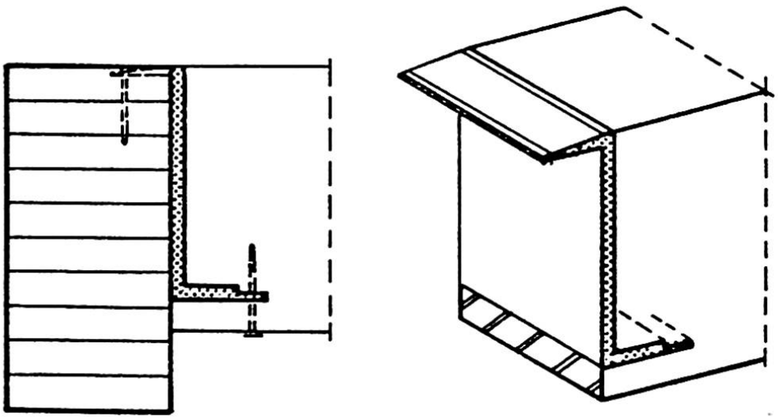

Fig.1
Special metal (aluminium) connector for a beam-to-beam connection.

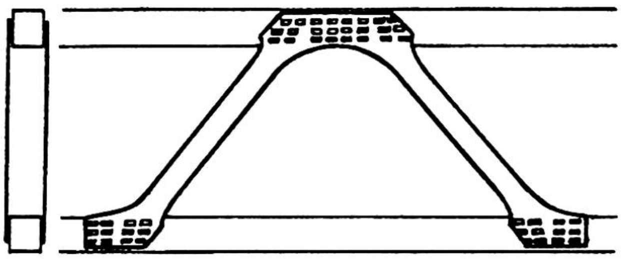

Fig.2
Truss beam with cold-formed steel plate diagonals integrated with nail plate connections.

4. BASIC ASPECTS ON WOOD DESIGN

4.1 Statistical aspects on the strength and stiffness of wood, fasteners and structures

For the practical application of probabilistic theories and for other purposes statistical data on wood strength and stiffness are needed. The basis for these data is of course an extensive testing. Simulation studies using good mathematical models of detailed structural behaviour may, however, show the influence of changes in different parameters. Since the scatter in statistical data for timber and wood based materials is mostly very large, many specimens have to be tested to give reliable estimates for the desired strength and stiffness properties. Computer simulations may therefore be an attractive alternative for economical reasons. With such simulation models the influence of different types of structural defects may be studied.

It is a well-known fact that the strength of a timber structural member is reduced when the size of the member increases. The basis for the study of size effects is the Weibull theory, which was established by the Swedish researcher Waloddi Weibull in the late 1930´s. It is desirable that this class of methods is made into a practical tool for engineers.

The behaviour of timber structures under service loading conditions has not attracted as much attention from the research engineers as the ultimate limit state. It is important, however, to make the design engineer aware of the statistical aspects on the stiffness of timber structures, such as load sharing between members in floors etc.

4.2 Reliability of timber structures

There have been some efforts towards a reliability-based design for timber structures. Papers which show applications of safety concepts and probabilistic methods to wood structures are welcome. It is especially desirable that the work is brought forward to a state, which makes the methods usable for the practising engineer. New developments include the so called Load and Resistance Factor Design (LRFD).

For a proper use of reliability concepts in wood structure design, a sufficient amount of basic data for statistical distributions of engineering properties is needed for different structural materials and elements such as gluelam beams, round timber poles etc.

4.3 Computer aided design (CAD) of timber structures

The computer has - at least to some extent - been introduced as a tool for the design of timber structures, for example in the form of some drafting systems. However, systems where design and drafting are integrated seem to have been developed only to a limited extent, for example in some special applications such as roof trusses. A rather fast development of computer aided design and manufacturing (CAD/CAM) applied to the field of timber construction is expected. This will mean that for a housing project a computer and a common data base will support both the design and the automatic drafting of trusses and timber framing as well as the manufacturing of the components. There is also a trend from component design to a complete 3-dimensional design. Therefore, papers that describe integrated CAD/CAM- or CAD-systems using interactive computer graphics for timber structures are most welcome.

5. STRUCTURAL COMPONENTS

5.1 Lumber and gluelam

Machine stress rating for structural lumber is now established in the timber industry of several countries.

For gluelam there has been a tendency in some countries to increase the thickness of the laminations. For example in Sweden there has been a recent increase from 33 mm thickness to 45 mm. Some attention has been drawn to special design problems such as gluelam beams with holes of different shapes and to different other stress concentration problems at sections where an abrupt change in beam depth occur.

5.2 Built-up beams and stressed-skin elements

Much effort is still spent to acquire better knowledge of the behaviour of different types of structural elements of wood or wood products, also for elements where the wood material is combined with other materials such as steel or lightweight concrete. Many such composite elements have been developed during the last decade. The majority of these elements are beams of different material combinations. Other important components belonging to this category are stress-skin elements intended for use for either walls, floors or roofs. A study of composite elements should include the influence of duration of load, moisture content and moisture content variation. Both the static and the dynamic performance is of interest for some of these elements. In the developments of new types of composite structural elements it is of vital importance to try to select the material in an optimal way, taking the special properties of each material into consideration.

Based on these principles a number of different standardized ('pre-engineered') composite beams have been developed during the last few years. Some are variants of earlier types of I-beams or box-beams with hardboard or plywood web and flanges of lumber or laminated veneer, Fig.3. A new type of small box-beam (or column) recently presented in Sweden has hardboard web and flanges of cold-formed thin-walled steel plate.

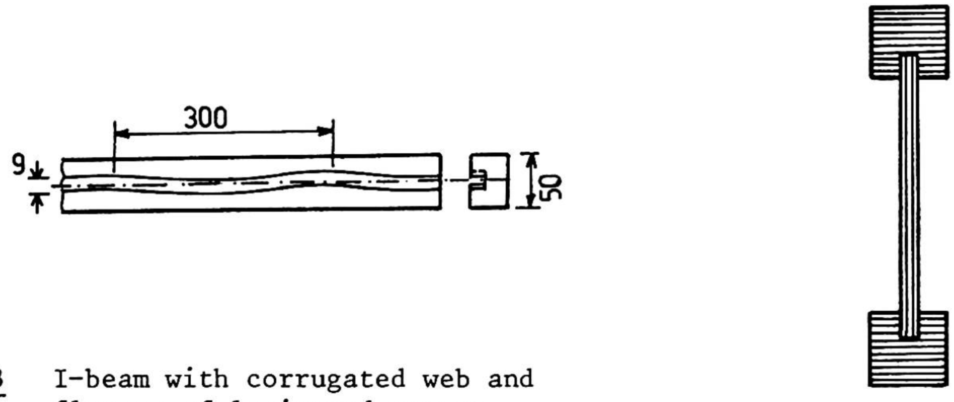

Fig.3 I-beam with corrugated web and flanges of laminated veneer.

Small standardized truss-beams with parallel flanges of lumber and falling and rising diagonals of steel rods were introduced in Sweden a couple of years ago. The steel rod is bent in a zig-zag pattern and placed in slots cut into the middle of the flanges. The product is marketed under the name of Wirewood and is intended to be used as wall studs, roof beams for small spans etc.

Another recent pre-engineered truss-beam with lumber flanges is shown in Fig.2. The diagonals are of thin-walled steel plate, cold-formed to give a cross section which is stiff enough to give sufficient safety against buckling. The ends of the diagonals are shaped as nail plates (tooth-plates) and the whole set of two diagonals and integrated nail plates shown in Fig.2 is pressed into the lumber flanges.

The following list of different stressed-skin elements, where timber and wood based materials work in complete, or partly, composite action with steel or concrete, just gives some examples :

- o Joists of timber or of built-up (wood and wood based panel) beam sections and steel plates,
- o Timber joists and concrete plate(s),
- o Thin-walled, cold-formed steel sections (trough or C-shape) in composite action with panels of wood based materials such as plywood or chipboard. Glued or screwed steel-to-panel connections.
- o Profiled steel joists pressed into chipboard flanges.

5.3 Columns

Some developments have been made that shed more light on the behaviour of simple columns. The influence of different inherent defects on column strength should be an interesting theme for a paper at the Congress.
For larger buildings built-up columns are sometimes used. The most popular column type for such buildings is the glued laminated column.

5.4 Trusses

Because the roof truss is a structural element which is mass produced for small houses a tremendous amount of work has been carried out in order to find better forms of truss design, especially concerning the joints. Research towards better understanding of the behaviour of a typical roof truss is still underway in different countries. This research concerns the full behaviour up to failure load. The computer has been introduced as a rather common design tool for this type of components.

Different trusses have been designed with combination of steel and wood, e.g. where some or all diagonals are steel rods or steel tubes. Some of these types are the small standardized roof trusses discussed in section 5.2.

5.5 Arches and frames

For larger halls and industrial buildings several types of arches and frames have been used for special purposes, thus showing the possibilities which the material offers the designer. Many of these buildings, but not all, are of gluelam.

5.6 Shells and other three-dimensional elements

Different types of folded plates and shells were reviewed by Möhler |1|. Among the structures of this category are space trusses which, although used quite a lot in structural steel, are less common for timber structures.

6. BEHAVIOUR OF STRUCTURAL SYSTEMS

There have been some recent developments in structural wood systems such as pre-engineered buildings and standardized framed buildings. A good designer should make use of the special advantages of wood (and wood based materials) as a structural material and try to minimize the influence of the disadvantages.

6.1 In-service behaviour

As mentioned in section 4.1 there has been a trend in recent years to pay more attention than before to the serviceability limit state. In earlier days the main interest in this state was for deformations under long term loading. The development of lighter and more efficient structures, however, has led to a situation, where problems concerning the dynamic behaviour of structures occur more frequently. Since the stiffness of the structure then becomes an important parameter, the static interaction between different components will be of great interest as a means of reducing the deflection. Design criteria which only contain 'static' parameters such as stiffness, however, can only be regarded as tentative until a complete dynamic criterion with parameters such as damping and mass has been etablished.

For example for light-weight, long span floors — both in small houses and for other applications — the vibrational behaviour due to different kinds of dynamic loads has been the object of recent investigations, see references in |7|. The dynamic performance of such floors is of interest with respect to the comfort or discomfort felt by the user |7|.

Studies concerning the interaction of the main parts of the whole 3-dimensional structural system of a building are still rare. For 2-dimensional subsystems such as walls, floors and roofs subjected to in-plane loading there is, however, some knowledge concerning deformations and deflections under service load, cf. Fig. 4.

6.2 Ultimate limit state

Apart from the improvement of traditional knowledge concerning the failure loads of wood structures subject to static loading there have been some trends also to look into the load carrying capacity under dynamic loading such as earthquake loading. Here questions concerning energy absorption may be of interest. The problem area ought to be touched upon at the congress, because earthquake resistant design is of such vital importance for many countries.

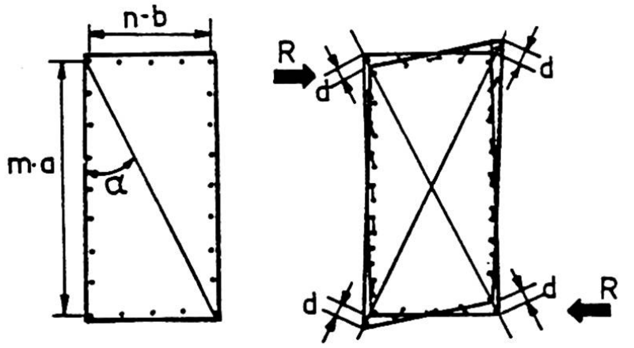

Fig.4

Nailed wall element of lumber and board plates. Deformations due to racking load.

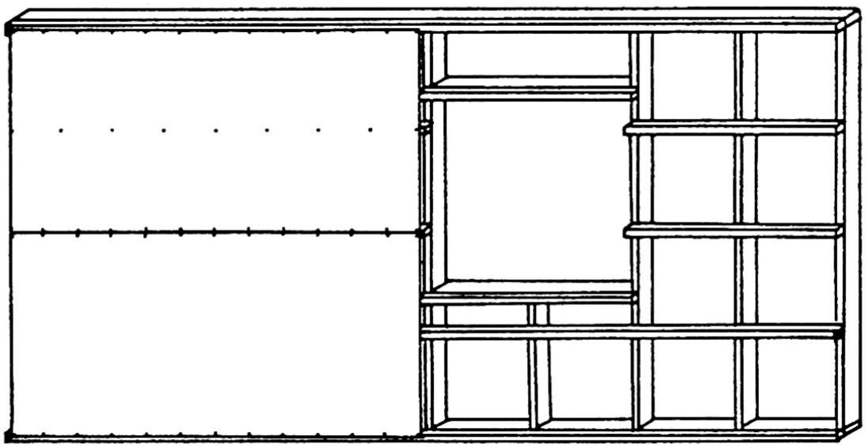

Fig.5

Timber-frame wall for low-energy consumption house.

6.3 Light-weight framing

There is an increasing need for better, low-energy consumption houses. This leads to a partly changed design also for wood houses. The 'energy efficiency' of the building becomes more important, which leads to a more general approach to the overall design problem including a systems approach to 'environmental design'.

A typical wall structure for a low-energy timber house is shown in Fig.5. Apart from the ordinary wall studs there are a number of horizontal battens to which plates of chipboard, hardboard, plywood or gypsum board are nailed. In this example wood material panels are used acting as a stressed-skin which stiffens the building against wind loading (wall-racking case). If the stressed-skin action is used both in walls, floors and roofs the weight of the building will be reduced. The stressed-skin elements may alternatively be designed as sandwich structures with a thick core of insulating material in composite action with the skin as was shown in a contribution to the 11th Congress |2|.

Another way of creating an efficient structure for a small house is by a series of parallel light-weight stiff timber frames, a system recently improved by developments at the US Forest Products Lab.

7. NEW STRUCTURES

Many interesting developments are continuously made in structural design utilizing the special properties of wood and wood based materials. These new designs are found in different types of structures in buildings, halls, bridges etc, Fig.6. What is wanted for the seminar are presentations of really new concepts and new solutions that deserve special attention from the practising structural engineer. Examples of new projects with structures of exciting new forms are found in buildings of a special character and buildings with long spans (halls for sport etc, churches, shopping centres, industrial buildings etc) and in bridges. Innovative designs may, however, also be found in small buildings like one-family houses, cf. section 6.3, or in simple small buildings like pole buildings, Fig.7.

Of special interest are papers which show the proper use of impregnated wood in different applications. Here, again, developments concerning pole structures ought to form a special subsection.

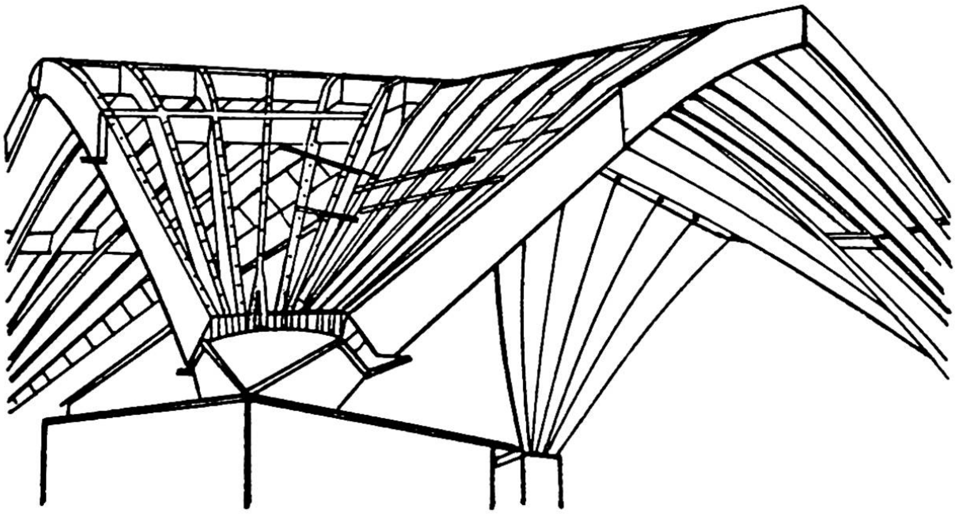

Fig.6 Glued laminated arches for a large hall.

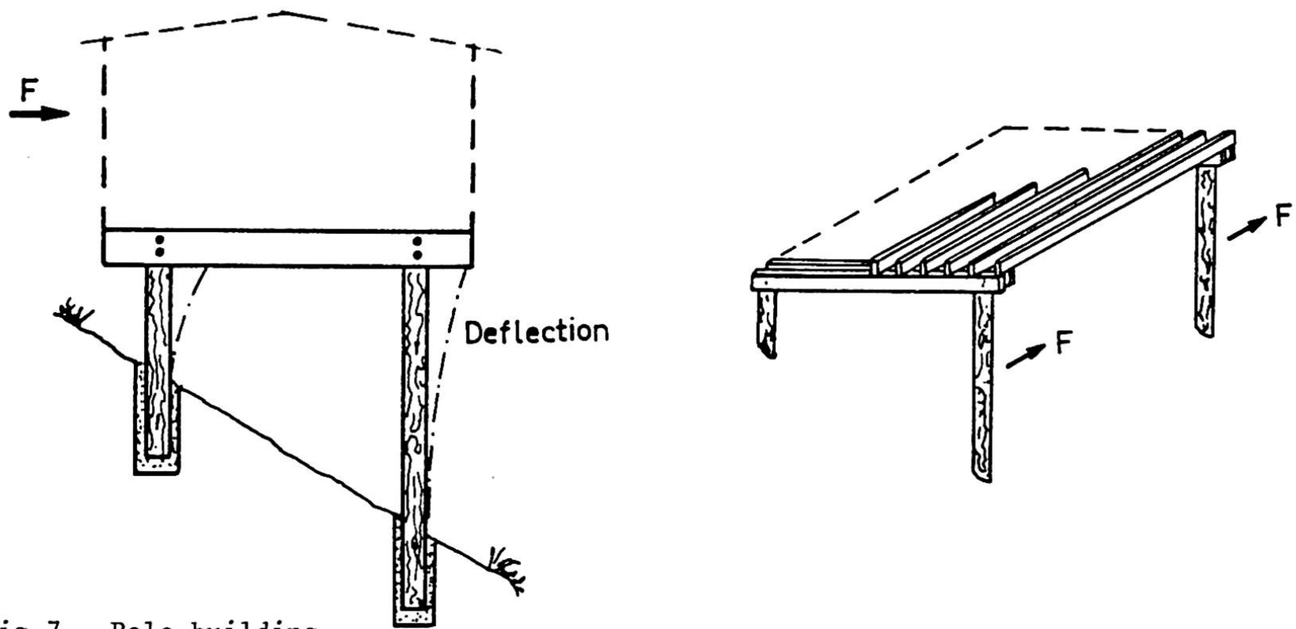

Fig.7 Pole building.

8. WOOD CONSTRUCTION, TRANSPORTATION AND ERECTION

As for the field of design there are continuously new developments concerning more efficient methods for the production, transportation and erection of timber structures. This is necessary in order to make modern timber structures competitive. Due to lack of space we will here only refer to the brief review by Möhler in |1|.

Apart from the construction of new buildings the rehabilitation and repair of existing structures also belongs to the field of wood construction. This kind of work is also of increasing importance to the structural engineer, and it is expected that - although there is an IABSE Symposium in Venice in September 1983 on the strengthening and repair of buildings - there will still remain some special questions within this field concerning timber structures that ought to be touched upon in Vancouver in 1984.

9. PROTECTION AND MAINTENANCE

A considerable amount of research and testing work has been invested on the problem of fire research during the past two decades. Therefore, there is now a basis for design rules for timber structures with respect to fire conditions. These refer both to fire *protection* and the behaviour of structures under fire. Some tentative design rules have been published in the CIB Structural Timber Design Code |5| by the Working Group CIB-W18.
Some special problems that ought to be treated in more detail concern (1) different timber structures under static loading combined with fire loading and (2) safety problems with respect to fire.

As mentioned before, e.g. in section 2.5, moisture performance of structures in different buildings is an important problem. *Impregnation* is one means for timber preservation as indicated above. It is, however, of vital importance that the designer is aware of different aspects of the moisture problem, since he may himself to a large extent contribute to the improvement of the resistance of the structure against fungi attack simply by designing with respect to the way moisture from the environment interacts with timber. The field of knowledge needed here may be called 'design for moisture protection' and it is a field that certainly deserves more attention by structural engineers.

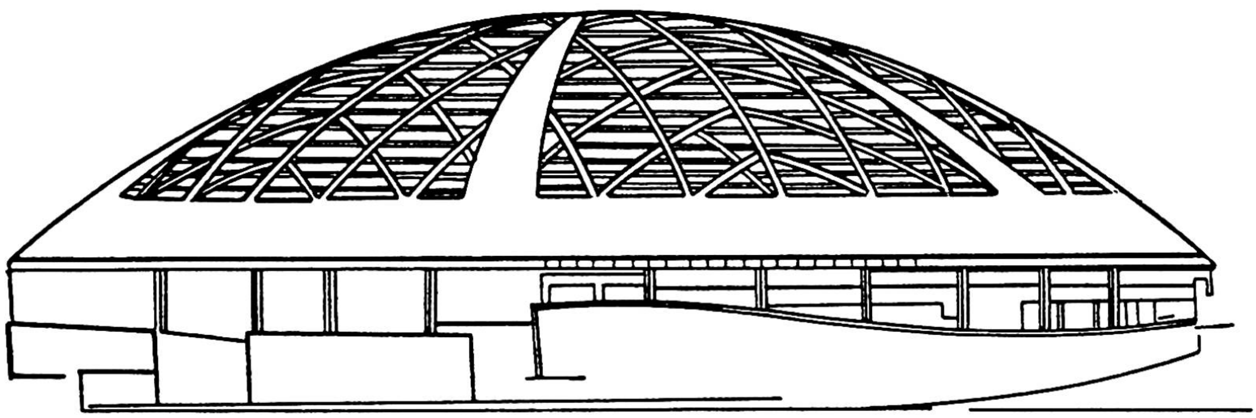

Fig.8 World's largest wooden dome, Tacoma Wash. Diameter just above 160 m.

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Seminar XI

Developments in the Construction of Reinforced and Prestressed Concrete Structures

Développements dans l'exécution de constructions en béton armé et précontraint

Entwicklungen bei der Ausführung von Stahlbeton- und Spannbetonbauwerken

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Bouygues
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1. Prestressed Bridges

For prestressed bridges, it is extremely important to carefully examine the methods of construction which are often determined under the constraint of finding ways to reduce costs. Economy constitutes a favorable factor when the contract is lump sum or paid according to a schedule of prices, in which case the quantities of materials used for the completed structure and those calculated for the chosen method of construction must be separated, and this separation is very difficult to establish.

Beyond this purely economic aspect, each structure reflects, at the very least, the history of its construction. Thus, the method of construction is highly important to the performance of the structure, or its quality, long after the construction stage itself.

In my opinion, it is this aspect of construction that would be of interest to discuss among specialists. It seems to me that an examination reaching beyond habits and the demand for economy, such as the problem of the wear of materials, would be highly interesting. A discussion of this type would certainly not challenge today's methods, none of which are inherently good or bad. But a general sharing of ideas would permit us to slowly sort out the best methods, and would surely generate new concepts.

This is where the interest in a large conference lies: to examine one's own ideas after hearing the opinions of colleagues on a subject in which they also are specialized.

These sessions might include discussions on:

1.1 totally cast-in-place bridges, cantilever erection, mobile formwork, traditional scaffolding parallel to the shore followed by rotation and keying, etc.

1.2 entirely precast bridges, cantilever erection, erection by entire spans, etc.

1.3 combined systems, such as bridges with precast girders and a cast-in-place upper deck, with or without prestressing,

1.4 composite structures (steel plus prestressed concrete), having, for example, steel webs. These structures could be either precast or cast in place.

The above summary of construction methods is far from complete. However, its impact on the choice of the structure itself is evident immediately.

Other aspects which should not be neglected in the discussions include the comfort of the users of the bridge, for example, taking account of the presence of multiple joints, the geometrical quality of the wearing surface, the aesthetics or even the easy possibilities for checking and maintaining the structure.

I would like to insist on the detailed analysis of the consequences of every nature resulting from the methods of construction. It would not be worthwhile merely to give descriptions. The analytical justification of ideas or precise observations on existing structures would be much more interesting.

2. Other Structures

All of the above applies to large roof structures, dams, nuclear containment buildings and storage tanks.

Although the problems are not always the same, the history and the methods of construction remain engraved on the structure.

Depending on the method, the stresses or the deformations of a structure can be very different in the initial stages of its existance.

Once again, in this case, I think highly explicit discussions on these topics would be very interesting.

In conclusion, I would like to say that although today we possess very powerful tools for structural analysis, they are of limited interest if the precise state of initial stresses and deformations of the structure are unknown. Therefore, it is necessary to carefully study the consequences of every construction method.

The following questions deserve to be answered:

Are some methods of construction more certain than others?

Are some methods more favorable to the performance of structures in use?

In addition, it would be profitable to discuss the safety aspects of different methods.